



Assessment of the reactivity of expansive soil in Melbourne metropolitan area

A thesis submitted in fulfilment of the requirements for the degree of Master of Engineering

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Declaration

I certify that except where due acknowledgement has been made, the work is that of the author alone; the work has not been submitted previously, in whole or in part, to qualify for any other academic award; the content of the thesis is the result of work which has been carried out since the official commencement date of the approved research program; any editorial work, paid or unpaid, carried out by a third party is acknowledged; and, ethics procedures and guidelines have been followed.

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ABSTRACT

Expansive or reactive soil refers to clay soils that undergo significant volume change in response to changes in soil suction. This volume change occurs as swelling upon wetting, and shrinkage upon drying. In Australia, the design of residential foundation slabs constructed on expansive soil is based on the estimated ground surface movement, y_s . To calculate y_s , it is essential to know the reactivity index.

In this study, a series of laboratory tests which include shrink-swell test, soil-water characteristic curve, liquid limit, plastic limit, linear shrinkage, soil suction measurement and X-ray diffraction (XRD) have been performed in the geotechnical laboratory of the School of Civil, Environmental & Chemical Engineering at RMIT University to assess the distribution and nature of expansive soils in the greater metropolitan area of Melbourne. More than 70 soil samples were collected from 47 field sites across 37 suburbs in Victoria, covering a range of geological/geographic conditions and areas designated for future residential development.

The results of the laboratory tests are used to establish a library of shrink-swell indices. An interactive reactive soil map for Melbourne has been developed, which provides (1) the site location, (2) depths of soil samples collected, (3) site geology information, (4) shrink-swell indices (I_{ss}) and (5) traditional index properties of soil (liquid limit, plastic limit, linear shrinkage). The interactive map and database can be used by practitioners to assess and calibrate the shrinkage index estimated on a visual-tactile basis.

The empirical equation for calculation of soil shrink-swell index and two assumptions (i.e. an empirical correction factor of 2 for axial swelling test and assumed suction change range of 1.8 pF) adopted in AS1289.7.1.1 (2003) were assessed experimentally in this study. The results suggest that the values of correction factor (for one-dimensional swelling) between 2.6 and 2.9 are likely to be appropriate. The analysis of soil suction data at effective saturation and air dry after shrink-swell tests indicates that a value of soil suction change within the range 1.8 to 3 pF may be more appropriate.

To evaluate the effects of surcharge pressure, sample size and initial water content on shrink and swell tests, further laboratory tests have been performed and the results show (1) an increase in surcharge pressure led to a decrease in swelling strain, (2) The swelling strain increased slightly with the diameter of soil samples and (3) the higher the initial water content, the lower the swelling strain and the higher the shrinkage strain.

In addition, a number of correlations were attempted in this thesis between the shrink-swell test and other soil tests including linear shrinkage, plastic index and liquid limit. The results show that there is no obvious correlation between the shrink-swell index and traditional soil indices such liquid limit, plastic limit, plastic index and linear shrinkage.

The main contributions of this research are (1) a database/interactive map which is intended to be a useful tool for design engineers and building industry and (2) evaluation of the empirical equation and assumptions introduced in the Australian Standard which can lead to a better understanding of shrink-swell test and subsequently improving the calculation of the shrink-swell index.

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NOTATION

ψ	total potential
ψ_{os}	osmotic potential
ψ_m	mastric potential
ψ_{ob}	overburden potential
ψ_g	gravitational potential
γ_d	dry density
γ_w	buck density
σ'	effective stress
μ_a	pore air pressure
μ_w	pore water pressure
S	the suction
S_t	total suction in Kelvin's equation
S_r	degree of situation
I_{pt}	instability index
I_{ss}	shrink-swell index
I_{ps}	shrinkage index
I_{cs}	core shrinkage index
I_{is}	loaded shrinkage index
e	void ratio
e_0	initial void ratio
m_0	initial mass of soil sample
V_0	initial volume of soil sample
w_i	initial water content
m_s	the mass of soil particles

m_b	mass of container + wet soil before put in oven
m_c	mass of container + wet soil after put in oven
m_t	mass of container
D_{ring}	soil sample ring diameter
V_s	the volume of soil particles
V_w	volume of water in the soil
V_v	volume of voids in the soil
V_{mo}	molecular volume of water,
$\frac{P}{P_o}$	relative humidity
R	universal gas constant,
T	absolute temperature
P	partial pressure of pore water vapour
P_o	saturation pressure of water vapour over a flat surface of pure water
y_s	surface movement
Δu	suction Change at Depth (z) from the surface
H_s	depth of site suction change
Z	depth from the finished ground level to the point under consideration in the uncracked zone
Δw	change in moisture content of the dry weight of the soil
ε	soil strain
ε_{sh}	the total shrinkage strain
ε_{sw}	the total swelling strain
D_0	distance between the rounded heads of the pins
D_d	distance between the rounded heads of the pins after oven dry

D_s	the total swelling of the sample
D_i	the initial settlement in swelling test
H_0	average initial length of the specimen
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index
L_s	longitudinal shrinkage of the specimen
L	initial length of the specimen
u_f	final suction
u_i	initial suction
h	soil sample ring height
α_{eff}	volume correlation factor
Δu_{eff}	range of soil suction change
σ	standard deviation
R^2	coefficient of determination

CHAPTER 1: INTRODUCTION

1.1 Background

Expansive soil (also referred to as reactive soil or shrink-swell soil) is any soil composed predominantly of clay that undergoes appreciable volume change following its moisture content change (soil suction). This volume change occurs as swelling upon wetting, and shrinkage upon drying, which in many cases can cause damage to lightweight structures such as roads, pavements and residential buildings.

Damage to lightly loaded structures founded on expansive soils has been widely reported in many countries such as Australia, China, India, Israel, South Africa, the United Kingdom and the United States of America ([Li and Cameron, 2002](#)). In China alone, total damage due to expansive soil is estimated to cost \$US 15 billion per year. The annual cost of expansive soil damage in the United States is estimated to be approximately \$US 15 billion, more than twice the damage from floods, hurricanes, tornadoes, and earthquakes combined. ([Li et al. 2014](#)).

The problems are particularly significant in Australia as approximately 20% of land area is covered with expansive soils and expansive soils exist in six out of eight of Australia's largest cities ([Delaney et al, 2005](#)). A map of the distribution of moderately to highly expansive soils is provided in [Figure 1](#). In Victoria, approximately 50% of the surface area is covered by expansive soils ([Richards et al, 1983](#); [Holland 1981](#)). Most Australian families own one- or two-storey detached dwellings which are constructed with masonry external walls that are intolerant of ground movement induced by non-uniform soil moisture changes due to rainfall and evaporation, garden watering, leaking water pipes, or tree root activity. Consequently, considerable research in Australia has been devoted to the ground movement prediction and the design of residential footings to cope with the ground movements due to the long term seasonal change in the soil suction profiles ([Li and Zhou, 2013](#)).

In Australia, the residential footing design and construction practices have been guided by the national standard AS2870 ([2011](#)) since 1986. As a routine practice, site classification is based on the characteristic surface movement over the life of the house, which is based on

the reactivity index of the soil, the design depth of soil suction change and the change in suction at ground surface. Central to the calculation of the characteristic surface movement are estimates of the instability index of the soil.

The Australian Standard AS2870 (2011) provides the following three methods for the estimates of the instability index:

- (a) Laboratory tests for soil reactivity, as set out in AS1289.7.1.1 (2003), AS1289.7.1.2 (1998), and AS1289.7.1.3 (1998);
- (b) Correlations between shrinkage index and other index tests for soil type; and
- (c) Visual-tactile identification of the soil by a suitable qualified and experienced person.

In Victoria, classification of reactive soil sites for residential construction is largely undertaken using the visual-tactile method. The soil profiles and soil layers are identified and values of shrinkage index are estimated for the different soils. The design surface movement is then calculated based on the design suction changes given in the AS2870. This method relies firstly on the experience of the engineers to identify a soil type and secondly on the uniformity of soil. However, the research conducted by Jaksa et al (1997) and Delaney et al. (2005) found that the visual-tactile method is highly inaccurate.

Australian Standard AS2870 (2011) introduced three tests to measure instability indices, namely shrink-swell test (AS 1289.7.1.1, 2003), loaded shrinkage test (AS 1289.7.1.2, 1998) and core shrinkage test (AS 1289.7.1.3, 1998). Compared to other two methods, the shrink-swell test has two distinct advantages: (a) both swell and shrinkage strains are considered so that the sample may be either very wet or very dry; (b) without the need to measure soil suction values.

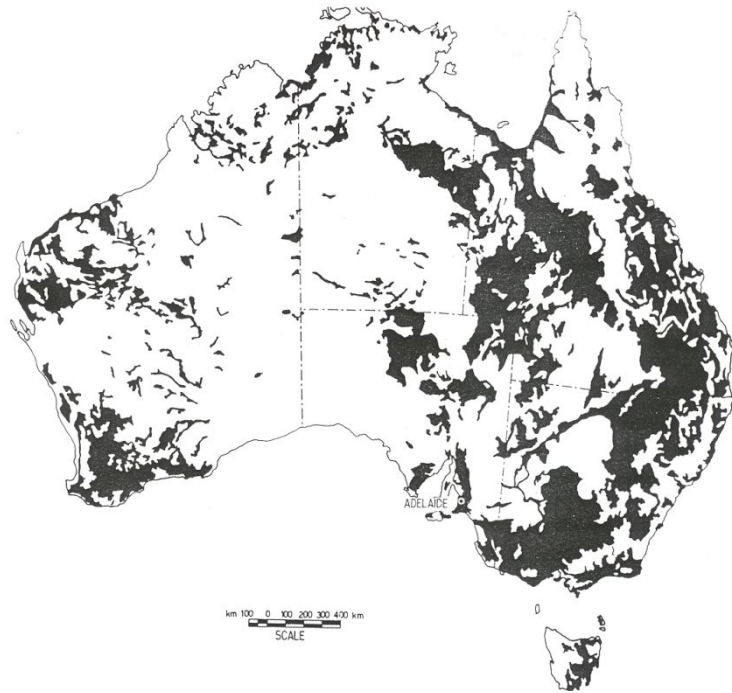


Figure.1 Distribution of moderately to highly expansive soils in Australia
(After Richards, 1984)

1.2 Research objectives

In this study, a series of shrink-swell tests have been conducted at the geotechnical laboratory of the School of Civil, Environmental & Chemical Engineering at RMIT to assess the reactivity of expansive soils in the metropolitan area of Melbourne. Soil samples were collected from 47 different field sites in 37 different suburbans covering a range of geological soils. Other laboratory tests that include Atterberg limits tests, linear shrinkage, grading, soil suction measurement, X-ray diffraction (XRD) test and scanning electron microscope (SEM) were also carried out.

The main objective of this research was to provide information about the nature and properties of the expansive soils around Melbourne area.

The specific objectives of this study are:

- to compile a library of shrink-swell indices which can be used to assess the values of I_{ss} estimated based on the visual-tactile method by the local geotechnical practioners;

- to assess the correlation between the shrink-swell indices and traditional soil indices such as plastic index, linear shrinkage and liquid limit;
- to investigate the effect of initial moisture contents (suctions) on the results of shrink-swelling tests;
- to investigate the effect of surcharge pressure on the results of swelling tests;
- to investigate the effect of sample size on the results of swelling tests;
- to evaluate the value of the lateral coefficient of expansive soil movement, α .
- to evaluate the suction change of 1.8 pF suggested by AS2870 (2011) for the calculation of shrink-swell indices.

1.3 Thesis arrangement

This thesis is divided into five chapters. A brief description of each chapter is given below.

Chapter 1 – Introduction

This Chapter provides an introduction and the main objectives of the research, and background information on expansive soils and the methods for estimates of the instability index. The thesis structure is also outlined in the Chapter.

Chapter 2 – Literature review

This chapter provides a detailed review of the Australian Standard for residential slabs (AS2870, 2011). Shrink-swelling test (AS 1289.7.1.1, 2003), the site classification methods and the characteristics and behaviours of expansive soils as well. Various types of damages to residential building caused by expansive movement are also discussed in this Chapter.

Chapter 3 –laboratory test

This Chapter describes how and where soil samples were collected. The procedure and methodology of laboratory test are given. The methods used for calculation of the values of shrink-swell index and the influence factors are also discussed.

Chapter 4 – Testing results and data analysis

This Chapter summaries all the laboratory test results, including shrink-swell test, Atterberg limits test, soil suction measurements, etc. Based on the shrink swell test results, a map of soil shrink-swell indices are developed for the Melbourne metropolitan area. The method for calculation of shrink swell index recommended by AS2870 (2011) is also reviewed. The effects of initial moisture content, different sample sizes and surcharge pressures are also evaluated. In addition, the value of the lateral coefficient of expansive soil movement (assumed as 2 in AS2978) and the range of suction change (taken as 1.8 pF in AS2870) are assessed as well.

Chapter 5 – Conclusion and recommendation

This chapter provides the conclusions of the research work and recommendations for future research.

CHAPTER 2: BACKGROUND AND LITERATURE REVIEW

2.1 Introduction

Australia has had national standards for site classification and design of residential footings for expansive soils for almost 30 years. This chapter introduces the Australia Standards which have been adopted in this research and presents the site classification process, current practice and laboratory tests to identify expansive clay. The background of the research is also presented.

2.2 Study area - Melbourne

Melbourne is the capital city of the state of Victoria and the second largest city in Australia. Melbourne has been named the world's most liveable city for the fifth year in a row in 2015, by the Economist Intelligence Unit's liveability survey of 140 cities ([ABC NEWS, 2015](#)). It lies on flat and undulating plains crossed by the Yarra River at the head of the broad and shallow Port Phillip Bay. Its city centre is situated at the northernmost point of the bay, near the estuary of the Yarra River. The metropolitan area extends south from the city centre along the eastern and western shorelines of Port Phillip, and expands into the Hinterland, toward the Dandenong and Macedon mountain ranges, Mornington Peninsula and Yarra Valley ([Australian Guide & Directory, 2015](#)).

The geology of the Melbourne region is characterised by its variable, widely diffused in the basement of the western portion of the area and part of the Mornington Peninsula. The area consists of folded bedrock, irregularly weathered and planted; the oldest rocks in the area are tightly folded marine sandstones, siltstones, and mudstones of the Ordovician period. Because rock type, slope morphology and weathering are all different in each region, soil behaviours and soil profiles are different.

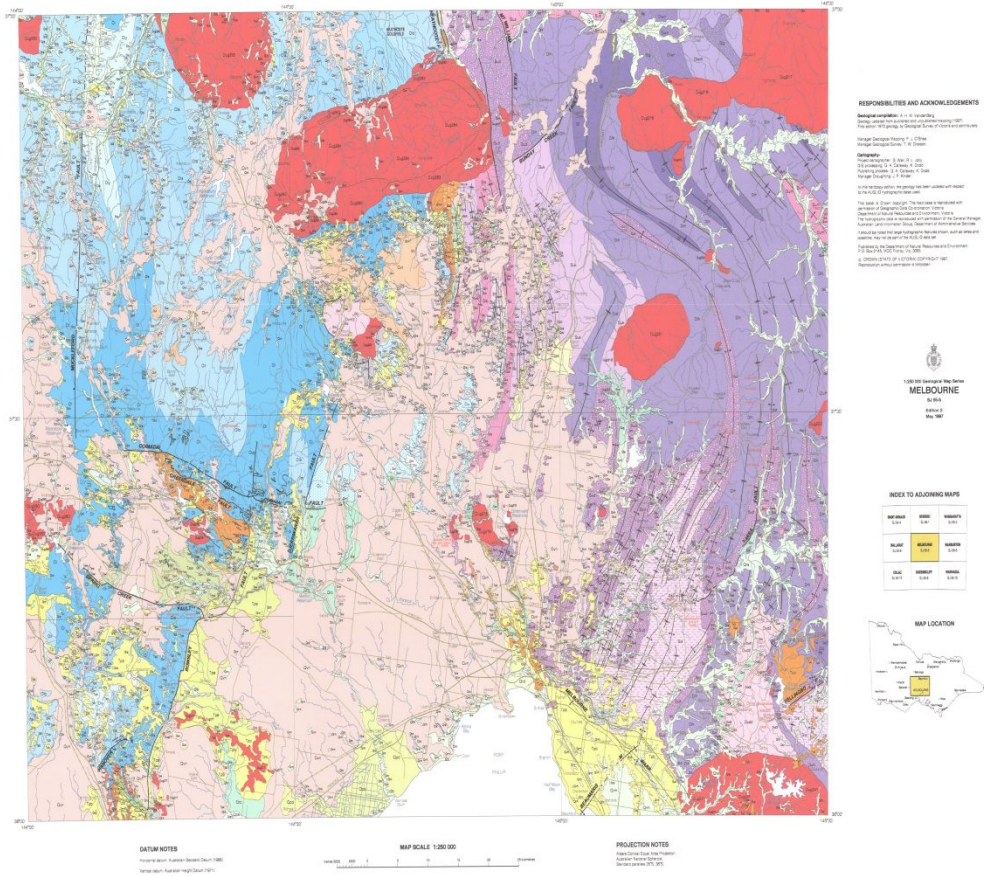


Figure.2 geology map of Melbourne
(VandenBerg, 1997)

This study mainly focused on the metropolitan region of Melbourne but some of the samples come from farther away, the sample collection area is bounded by Geelong to the west, Kilmore to the north, Warragul to the east, and Cowes to the south.

2.3 Soil classification system

2.3.1 Size distribution

Soil classification system is the system that geotechnical engineers use to classify and describe the soil by grouping together soils with similar behaviour and properties. It can be generally divided into two main groups, one for engineering purposes and another for soil science. For engineering purposes, the following are the most used classification systems (Garc a-Gaines & Frankenstein, 2015):

(1). Unified Soil Classification System (USCS)

- Based on particle size distribution, liquid limit, soil plasticity, and organic matter concentrations
- Widely used by geotechnical engineers

(2). United States Department of Agriculture (USDA) textural soil classification

- Based on particle size distribution
- Commonly used because of its simplicity

(3). American Association of State Highway and Transportation Officials (AASHTO)

soil classification

- Based on particle size distribution and soil plasticity
- Used mostly by state and county highway departments

Figure 3 present a comparison of particle size scales between the USDA, USCS, AASHTO, and others soil classification systems.

U.S.D.A.	CLAY	SILT		SAND					GRAVEL			COB-BLES	STONES
		fi.	co.	v.fi.	fi.	med.	co.	v.co.	fi.	med.	co.		
		.002		.05				2				76	250mm
INTER-NATIONAL	CLAY	SILT	SAND				GRAVEL			STONES			
			fi.		co.								
			.002		.02			2		20mm			
UNIFIED	SILT OR CLAY			SAND				GRAVEL			COBBLES		
				fi.	med.	co.		fi.	co.				
				.074				4.76			76mm		
AASHTO	CLAY	SILT	SAND				GRAVEL OR STONES			BOULDERS			
			fi.	co.			fi.	med.	co.				
			.005		.074		2			76mm			
PHI SCALE													
		.00195	.0078	.031	.125	.5	2	8	32	128	512mm		

Figure.3 Comparison of particle size scales between different soil classification systems
(Garc a-Gaines & Frankenstein, 2015)

For geotechnical works in Australia, the soil classification is mainly based on the standard AS1289.3.6.1 (2009). From Table 1, it can be seen that the classification system recommended by AS 1289.3.6.1 (2009) is very similar to the USCS system.

Expansive soil is composed predominantly of clay soil, i.e. the soil with particle size less than 0.002 mm.

Table.1 Australian standard of soil classification

Name	Sizes
Very coarse sand:	2.0-1.0 mm
Coarse sand:	1.0-0.5 mm
Medium sand:	0.5-0.25 mm
Fine sand:	0.25-0.10 mm
Very fine sand:	0.10-0.05 mm
Silt:	0.05-0.002 mm
Clay:	< 0.002 mm

2.3.2 Soil particle size distribution

Soil particle size distribution is a fundamental physical property of a soil and is usually presented as percentage of the total dry weight of soil occupied by a given size fraction. The soil's swelling potential largely depends on the total amount of clay mineral particles (particles with a particle size smaller than 2 μm) in the soil. Aspects such as water holding capacity, soil structure, and bulk density are all affected by the percentage of clay mineral particles in the soil. The particle size distribution of sand (0.05-2.00 mm) is usually determined by a sieve analysis where a dry soil sample is taken through different diameter sieves (from large to small), then the mass of soil retained in each sieve is determined and calculated as percentage of the total dry soil sample.

Smaller particles are usually determined by the hydrometer method or laser diffraction method. The hydrometer method uses a hydrometer which has a graduated stem and weight bulb, to measure the specific density of the suspension. The specific density depends on the weight of soil particles in the suspension at the time of measurement. The hydrometer method is time consuming, especially for the determination of the soil particles with a size less than 2 μm . It requires relatively large samples (at least 50 g) and gives unreliable results for particles having a size less than 1 μm . The principle of the laser diffraction method is that particles of a given size diffract light through a given angle. The angle of diffraction is inversely

proportional to particle size, and the intensity of the diffracted beam at any angle is a measure of the number of particles with a specific cross-sectional area in the beam's path. The laser diffraction method offers a number of advantages such as high level of precision, a fast speed of response, high potential for the repetition of results and a wide measurable particle diameter range. (Barth, H.G., 1984)

2.4 Material properties

2.4.1 Soil particle

Soils are made of mineral grains. Grain size distribution is very important in soil classification systems, as it determines some of the soil properties and behaviours. There are three main categories: spherical, angular, and rough. Differences in grain shape can reveal the reason why soils behave differently, such as most clay minerals are present in a platy shape.

2.4.2 Soil structure

Clay minerals are the product of many years of chemical and physical weathering. As rocks break down to small fragments, and through the weathering process over time, their pieces become small enough to be called clay. The different types of clay minerals determine the differences between clays.

Soil textural analysis is a key component of any minimum data set used for assessing soil quality. The clay material refers to hydrous aluminium phyllosilicates minerals with sheet-like structures and very high surface areas (Cameron, 1992). At molecular level, clay consists of two basic sheet structures as shown in Figure 4:1) Silicon - Oxygen Tetrahedron

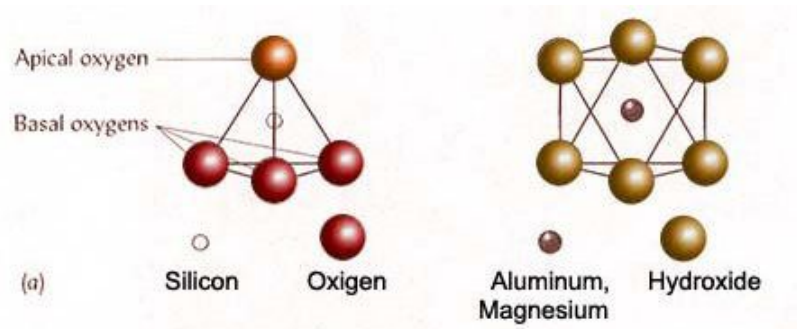


Figure.4 Clay sheet structure

(Centre for Teaching, Learning & Technology, UBC 2003)

2.4.2.1 Tetrahedral sheets

Tetrahedral sheets consist of central silicon sharing three out of every four oxygens. They are arranged in a hexagonal pattern with the basal oxygens linked and the apical oxygen pointing up/down.

2.4.2.2 Octahedral sheets

Octahedral sheets consist of individual octahedrons that share edges composed of oxygen and hydroxyl anion groups with Al^{3+} , Mg^{2+} , Fe^{3+} or Fe^{2+} serving as the coordinating cations, and surrounded by six O^{2-} , OH^{1-} ions, resulting into a tetrahedron show in Figure 5.

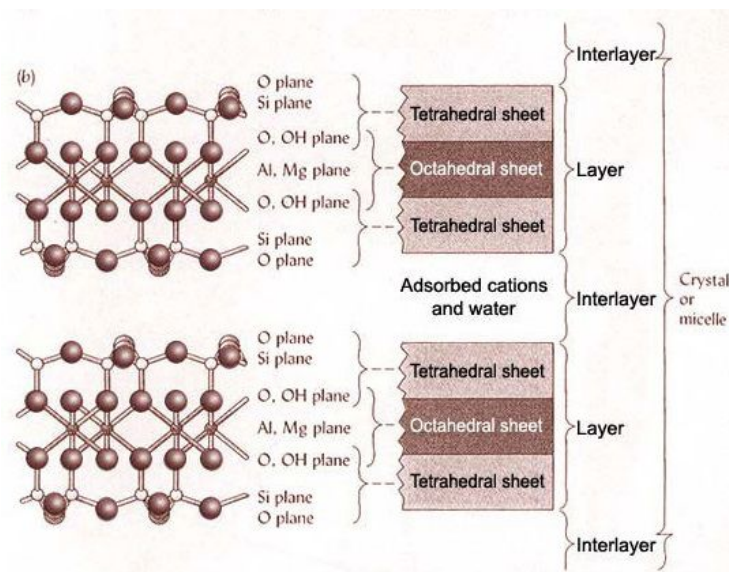


Figure.5 Phyllosilicate Clay Minerals
(Centre for Teaching, Learning & Technology, UBC 2003)

2.4.2.3 Structure units

There are two types of structure units formed in the soil, classified based on the permutation and combination of tetrahedral and octahedral sheets. When one tetrahedral sheet combines with one octahedral sheet, a two-layer structure of 1:1 clay minerals is formed.

Otherwise, when one aluminium octahedral sheet is sandwiched between two silicon tetrahedron sheets, they form a three layers structure of 2:1 clay minerals.

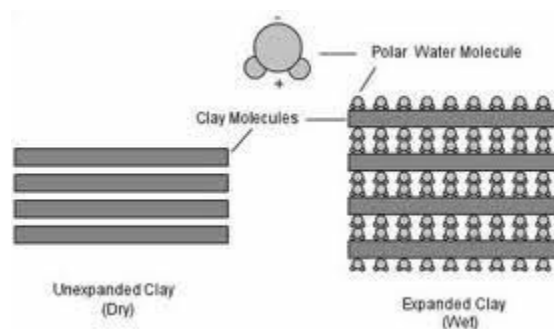
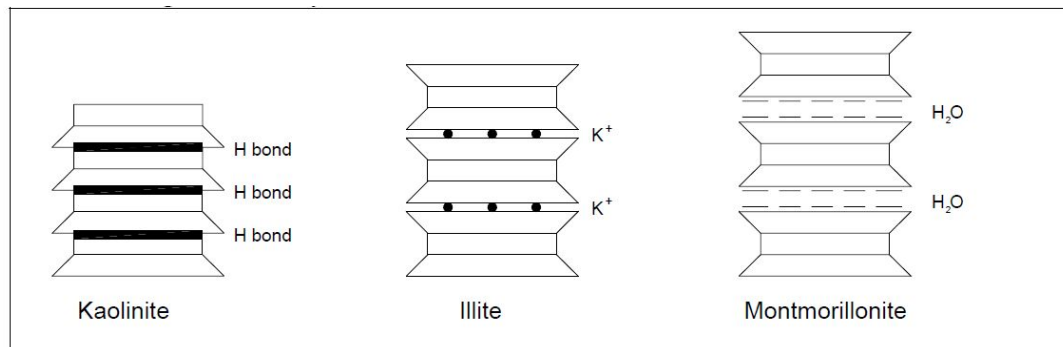


Figure.6 Chemical structure of unexpanded and expanded soil
(From: [Expansive Soil- Arkansas Geological survey](#))

The basic structure of 2:1 clay minerals is weakly bound to another 2:1 layer, although they are easily separated and allow water and contaminants to enter between them. Unlike the 2:1 type mineral, clay units are bound together by hydrogen, which has a stable structure and does not attract positively or negatively charged solutions (such as water). Therefore, the stabilisation at the interlayer, or the space between the sheets, determines the major difference between 2:1 and 1:1 clay minerals. Kaolinite and chlorite are typical examples of 1:1 clay minerals; smectite is an example of 2:1 clay minerals.

2.4.3 Laboratory test for soil texture

X-Ray diffraction test

Minerals in the soil are very important; -the different minerals in soil could cause large differences in soil behaviour, some minerals have significant influence on soil behaviour;

therefore the mineralogy of soil plays a very important role in soil mechanics. In the laboratory, the X-ray diffraction (XRD) test is one way to identify the minerals in the soil.

The XRD test is used for the testing of mineralogical composition, crystallographic structure, chemical composition and physical properties of clays. The results of this test could provide information about reactive clay mineral types and their composition in percentages.

2.5 Soil suction

Soil suction can be defined as free energy or relative vapor pressure. It could be explained as “the soil’s affinity to water.” This definition draws upon the notion that reactive soils can create movement beneath the surface, with direct correlation to their moisture content. This moisture content in terms of soil suction can refer to negative pressures generated – in turn causing major issues and headaches for home-owners, builders and engineers. There are three types of water kept into the soil:

- Gravitation water: Water free to move downward due to gravity, or water that drains from soil
- Capillary water: Water held in the capillaries or pores of the soil
- Hygroscopic water: Moisture that remains after capillary and gravitational waters are removed. This moisture is in the form of a thin layer that coats each grain of soil.

2.5.1 Total suction

Total soil suction (total water potential), ψ is defined in terms of the free energy or relative humidity of soil moisture:

$$\psi = -\frac{RT}{v_{wo} w_v} \ln \frac{\bar{u}_v}{u_{v0}} \quad \text{Equation. 1-a}$$

For pure water at 20 °C:

$$\psi = -135022 \times \ln \frac{\bar{u}_v}{u_{v0}} \quad \text{Equation. 1-b}$$

Where ψ is in kPa.

The total suction consists of four components under the heading of total water potential:

$$\psi = \psi_{ob} + \psi_m + \psi_{os} + \psi_g \quad \text{Equation. 2}$$

- = total potential = total Gibbs free energy
- = the overburden potential
- = the gravitational potential
- = the matric potential
- = the osmotic potential due to excess solute in bulk solution.

(overburden potential) and (gravitational potential) are two small components (<10 kPa) and can be ignored. Therefore it is commonly accepted that the total soil suction consists of matric suction ($\mu_a - \mu_w$) and osmotic suction (ψ_{os}):

$$\psi = (\mu_a - \mu_w) + \psi_{os} \quad \text{Equation. 3}$$

2.5.2 Osmotic suction

Osmotic (or solute) suction is a measure of the potential to build water in the soil due to the osmotic effects of the solutes in the bulk soil solution. This type of suction refers to the potential of the soil and water to bind due to osmotic effects of the solutes in the bulk soil solution. So this means that for a soil with some concentration of NaCl – sodium chloride - (salt) in its pore fluid, there is seemingly the potential to pull or suck the fresh water into the soil or to hold onto the water that already exists within the soil, if the ion concentration in the pore water changes, this can result in changes in the osmotic potential; typically some sort of contamination may cause this.

2.5.3 Matric suction

The matric suction is the difference between pore water pressure (u_a) and pore air pressure (u_w):

$$\text{Matric Suction} = (u_a - u_w) \quad \text{Equation. 4}$$

When a meniscus forms at the soil-air interface it reduces the vapour pressure in the water as the degree of saturation decreases. Furthermore, when soil particle size decreases relating to the soil pores reducing too, it leads the radius of the curvature to decrease and consequently the matric suction pressure. Unsaturated soil mechanics normally have negative pore-water pressures, while saturated soil mechanics have positive pore-water pressures.

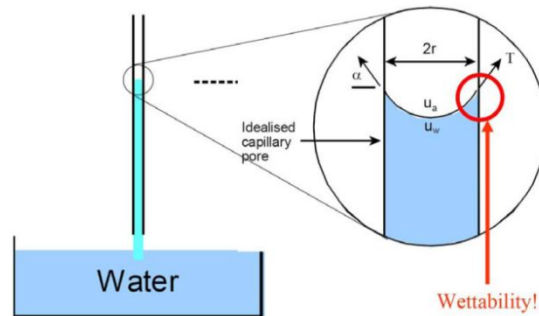


Figure.7 Illustration of capillary rise in a thin glass tube

2.5.4 Suction change in soil profile

In a typical unsaturated soil profile, there are three distinct zones, as shown in the [Figure 8](#)

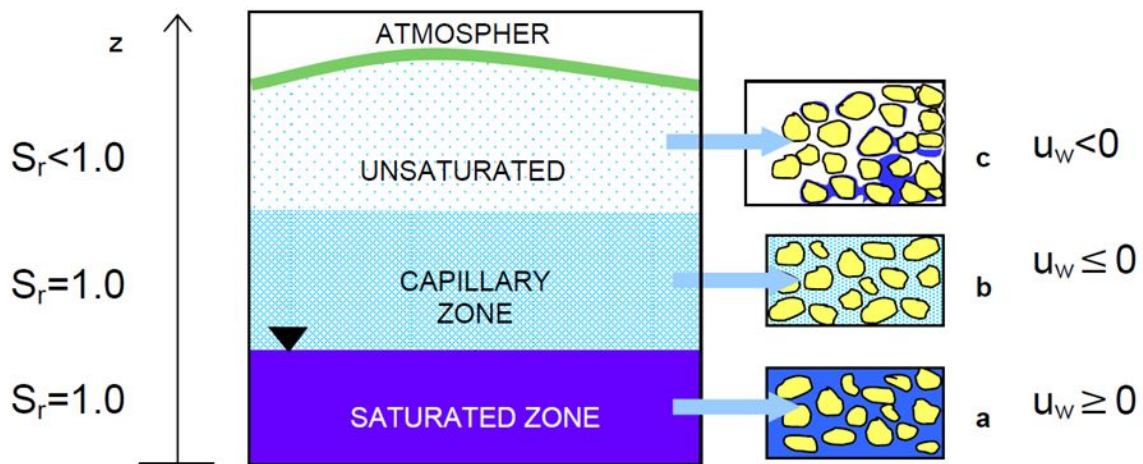


Figure.8 Typical unsaturated soil profile

In Figure 8, it can be noted that a soil layer profile has various sub-components that can be categorised in different zones. These zones are divided up based upon the expected/typical moisture or saturation of the soil within that layer, and can then be categorized and analysed separately. The location of the water table plays a significant role in the differences between the zones.

Generally, there are three main layers below the surface:

- Saturated zone: This zone should be below the underground water table; where soil is fully saturated, and there is no water flow. The pore water is continuous with positive pressure ($\mu_w > 0$) potential under positive pressure at equilibrium due to hydrostatic pressure.
- Capillary zone: This zone is the saturated zone above the ground water table, where continuous water is drawn up into soil pores though capillary suction. The process could lead to the negative pressure ($\mu_w < 0$) potential happening in pore water.
- Unsaturated zone: This zone is close to the ground surface, and is above the capillary zone. Soil becomes desaturated where air and water occupy the pore spaces; the water may become discontinuous and has large negative pressure potentials ($u_w < 0$).

2.5.5 Soil suction unit

As soil suction can be thought of as an internal soil pressure, it must therefore contain units of pressure. These pressure values that are obtained can sometimes be very large, therefore a logarithmic unit is used. Typically the unit “pF” is used and can be derived from the following formula:

$$pF = \log_{10} \left(\frac{s}{\gamma_w} \right) \quad \text{Equation. 5}$$

Where:

s is the suction

γ_w is the density

Soil suction can also be represented in $\log (kPa)$ unit system ([Fredlund and Rahardjo, 1993](#)). The relationship between the kPa and pF systems is approximately as follows:

$$\log_{10}(kPa) = \text{Suction in pF} - 1 \quad \text{Equation. 6}$$

$$\text{Suction in } pF = \log_{10}(|\text{suction in cm of water}|).$$

Table.2 Relationship between various suction units used

Height of Water Column(cm)	pF	PSI	Kgf/cm ²	kPa	Moulds	Atmospheres
1	0	0.0142	0.001	0.0981	0.00098	0.00097
10	1	0.1422	0.01	0.981	0.0098	0.00968
10 ²	2	1.422	0.1	9.81	0.098	0.0968
10 ³	3	14.22	1	98.1	0.981	0.968
10 ⁴	4	142.2	10	981	9.81	9.68
10 ⁵	5	1,422	10 ²	9,810	98.1	96.8
10 ⁶	6	14,220	10 ³	98,100	981	968
10 ⁷	7	105	10 ⁴	981,000	9,810	9,680
Note: Typical units of suction values may range from zero to over 15,000 Psi. These high values include a predominant component of osmotic suction.						

2.5.6 Soil-water characteristic curve (SWCC)

The Soil-Water Characteristic Curve, (SWCC), defines the relationship between the water content of a soil and soil suction. The SWCC can be used for obtaining many unsaturated soil property functions such as volume-change characteristic, permeability and shear strength functions (Li et al 2007). Additionally, the Soil-water characteristic curve can indicate numerous other soil properties.as noted in [Figure 9](#)

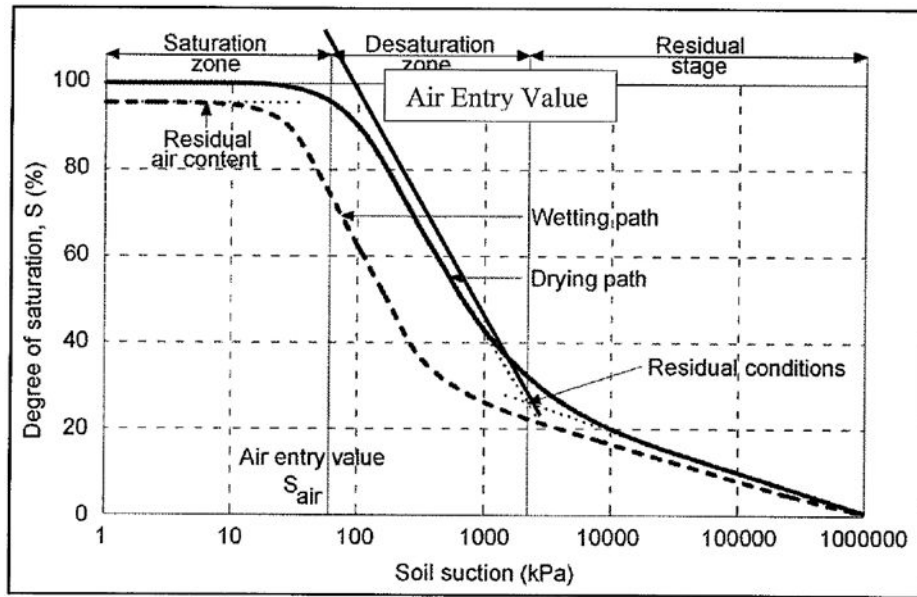


Figure.9 Typical Soil-Water Characteristic Curve (SWCC)
(Sillers, W. S., & Fredlund, D. G. 2001)

The SWCC is usually plotted on a log scale due to the great range of suction that can be experienced by the soil in the field. Three zones can be observed from a SWCC curve.

- 1) The saturation zone (also called capillary saturation zone), where soil is fully saturated due to capillary force in tension;
- 2) Desaturation zone, where soil suction value is exceeded by air entry value (air starts to come into soil, the degree of saturation start decreasing, and suction keeps increasing);
- 3) The residual stage, where soil is oven dry conditions and suction could reach 10^6 kPa (Cronney and Coleman, 1961).

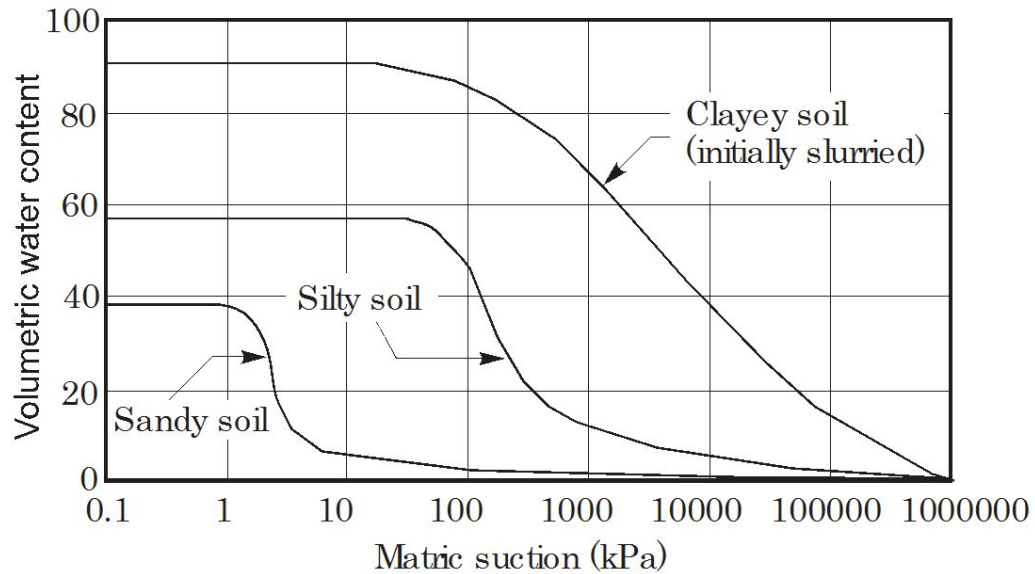


Figure.10 Typical SWCC for different soil types
(Fredlund and Xing, 1994)

The soil types and the SWCC could also be significantly related to the shrinkage and swelling behavior of soils. Figure 10 presents typical SWCC for different soil types. It can be seen that clay soils tend to be much more reactive than silts and sandy soils due to their fine particles and clay minerals, which result in a higher saturated moisture content and suction range.

2.5.7 Degree of saturation

The degree of saturation (S_r) is the ratio of liquid (such as water) to the total volume of voids in a porous material (such as soil), related to water content. It has a limit range between 0 and 1.0. The degree of saturation (S_r) can be calculated through the following equation:

$$S_r = \frac{V_w}{V_v} \quad \text{Equation. 7}$$

Where

V_w = the volume of water in the soil

V_v = the volume of voids in the soil.

As per the definition of saturated soil, the voids in fully saturated soil are full of water, which cause its degree of saturation to be always equal 1.0. After soil is oven dried, the water in soil is almost zero so the degree of saturation can be taken as 0.

2.5.8 Soil Suction Measurements

There are a few different methods that can be used to measure soil suction in the laboratory. In this study the suction is measured by Psychrometer (Wescor HR-33T), Dew Point Potential Meter (WP4) and the filter paper test to determine the suction of soil. All the three methods use psychrometric law (Fredlund and Rahardjo, 1993) by measuring the relative humidity to calculate soil suction. The thermodynamic relationship between total suction and relative humidity is given in Equation. 8. Total suction can be calculated using Kelvin's equation, which is derived from the ideal gas law using the principles of thermodynamics and is given as:

$$h_t = -\frac{RT}{V} \ln\left(\frac{P}{P_o}\right) \quad \text{Equation. 8}$$

Where

h_t = total suction,

R = universal gas constant,

T = absolute temperature,

V = molecular volume of water,

P / P_o = relative humidity,

P = partial pressure of pore water vapour

P_o = saturation pressure of water vapour over a flat surface of pure water at the same temperature.

When the temperature is 25°C, the following total suction and relative humidity relationship can be obtained:

$$h_t = 137182 \times \ln\left(\frac{P}{P_0}\right) \quad \text{Equation. 9}$$

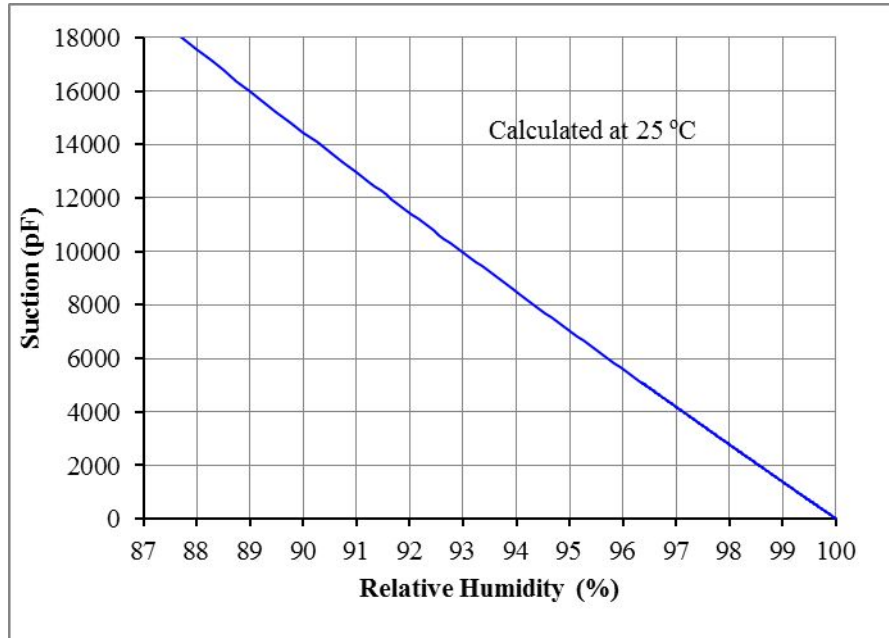


Figure.11 Total suction and relative humidity relationship

A plot of the Equation 9 is shown in [Figure 11](#) and it can be seen that total suction decreases as relative humidity increases.

2.5.8.1 Psychrometer (Wescor HR-33T)

Psychrometric technique is widely used in the laboratory to measure the low range soil suction. Psychrometer (Wescor HR-33T) is used to measure the total suction of a soil. The HR-33T Dew Point Microvolt meter is a self-contained electronic system that has been specifically designed for measuring water potential with thermocouple transducers. It contains a sophisticated sensing and control circuitry that automatically maintains the temperature of the thermocouple junction at the dew point temperature when operating in dew point mode.

Essentially what is occurring is a pattern of current is being fed into the chamber which maintains the junction contact at the temperature at which moisture neither evaporates from,

nor condenses onto the junction. This is known as the dew-point temperature. A change in the voltage indicates the dew-point temperature which may be appropriately calibrated against relative humidity and suction level. The measurements for the dew-points are taken on very small sub-samples, and the equilibrium period and reading time are relatively quick. The Psychrometer (Wescor HR-33T) can measure the total suction between 100 and 8000 kPa . The instrument needs to be calibrated using the known total suction of solutions.



Figure.12 Wescor HR-33T

2.5.8.2 WP4 Dew Point PotentiaMeter

WP4 Dew Point PotentiaMeter uses the chilled-mirror dewpoint technique. It measures water potential by determining the relative humidity of the air above a sample in a closed chamber (an AOAC-approved method). At temperature equilibrium, relative humidity is a direct measurement of water potential. The result can be determined immediately. The whole measurement only takes 5 to 10 minutes. WP4 measures total suction from 0 to -300 MPa with an accuracy of 0.1 MPa. Its working principle is similar as Wescor HR-33T, i.e. the relative humidity in the sample determines the suction of soils.



Figure.13 Water potential measurement

2.5.8.3 Filter paper method

Soil suction is related to soil water content through water retention characteristic curves. A number of methods have been developed to measure the soil suction. The filter paper method is an inexpensive and relatively simple laboratory test method and it has been accepted as an adaptable test method for soil suction measurements. The total suction and matric suction can both be measured with this method. Additionally, osmotic suction can also be easily calculated with the suction equation, but it does not allow to continuously monitoring the soil moisture changes.

The filter paper method measurement was developed in Europe in 1920, and Dr Gardner brought this method to the United States in 1937. It soon became a popular and basic suction measurement method used in soil investigation. Numerous researchers have used this method, such as Fawcett and Collis-George (1967), Al-Khafaf and Hanks (1974), Hamblin (1981), Chandler and Gutierrez (1986), and Houston et al. (1994).

Originally, the filter paper test only had one calibration curve for both matric and total suction, but soon it was reported that the total and matric suction calibration curves were not compatible. This simply implies that two different calibration curves, one for matric and one for total suction, need to be used in soil suction measurements. The calibration for the suction wetting curve for filter paper using salt solutions is based upon the thermodynamic relationship between total suction (and osmotic suction) and the relative humidity resulting from a specific concentration of a salt in distilled water. NaCl was selected as an osmotic suction source for the filter paper calibration.

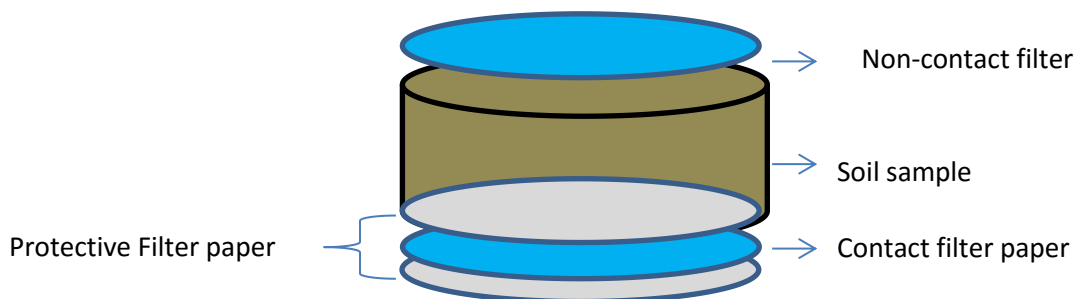


Figure.14 Samples of the filter paper.

2.6 Methods of estimating characteristic surface movement

Since 1986, Australian footing design and construction practices have been guided by a national standard. The current Australian Standard for residential slabs and footings is the 2011 edition (AS2870, 2011). Common to the three versions of the Standard, sites are classified according to soil profile and regional climate influence on soil moisture state. Site classification is based on y_s (refer to Table 3), the predicted design site surface movement, over the life of the house, which is based on design soil suction change profiles for different climatic regions of Australia (Li and Zhou, 2013). Central to the calculation of y_s are estimates of the instability index of the soil. In AS2870 (2011), the instability (or reactivity) index (I_{pt}) is defined as the percent vertical strain per unit changes in suction, taking into account the expected values of the applied stress, degree of lateral restraint and the soil suction range. The instability index is not a constant for a particular clay, but it can be estimated from the soil shrinkage index I_{ps} . The soil shrinkage index may be obtained using one of the following methods:

1. Correlations between shrinkage index (I_{pt}) and other clay index tests for soil types;
2. Visual-tactile identification of the soil by a qualified and experienced engineer.
3. Laboratory tests for soil reactivity, as set out in AS1289.7.1.1, AS1289.7.1.2. and AS1289.7.1.3;

2.6.1 Correlations between shrinkage index (I_{pt}) and other clay index tests

The Atterberg limits can be used to distinguish between different types of silts and clays. For most engineering purposes, the properties of soils can be defined by Atterberg limits such as plastic limit, shrinkage limit and liquid limit. The correlations between the shrinkage index (I_{pt}) and Atterberg limits have been studied by a number of researches. Mitchell and Avalue (1984) and Cameron (1989) found linear shrinkage to be a reasonable indicator of soil reactivity. However Delaney et al (2005) found there was no reliable relationships that exist between shrinkage index (I_{pt}) and Atterberg limits.

2.6.1.2 Visual-tactile method

The second approach is the visual-tactile method. In Victoria, classification of reactive soil sites for residential construction is largely undertaken using the visual-tactile method. In routine practice, the soil profile and soil layers are identified and values of shrinkage index are estimated for the different soils. The design surface movement is then calculated based on the design suction changes given in the Australian Standard AS2870 “Residential Slabs and Footings – Construction”. This method relies, firstly, on the experience of the engineers to identify a soil type, and secondly on the uniformity of behaviour of soil. However, the research conducted in the Newcastle-Hunter region (Delaney, et al, 2005) found that the visual-tactile method is highly inaccurate because of the diversity and abruptness of the local geological conditions, and large variability in the colours and textures observed in local clays either weathered from the same parent rock unit or a different lithotype.

2.6.1.3 Laboratory tests for soil reactivity

For the third method, shrinkage index, I_{pt} , can be determined from three laboratory tests: (a) shrink-swell test (AS 1289.7.1.1); (b) loaded shrinkage test (AS 1289.7.1.2); and (c) core shrinkage test (AS 1289.7.1.3). The advantages and disadvantages of the above three methods will be discussed in the follow chapter.

Jaksa et al (1997) found that the values of I_{pt} estimated by using the visual-tactile method varied by as much as 166% and were highly classifier-dependent. As a result, it was concluded that visual-tactile method was highly inaccurate, particularly when engineers/classifiers had limited experience and the estimations were not regularly checked against laboratory testing. The new edition of AS 2870 – 2011 now requires that the estimated I_{pt} be calibrated against laboratory testing at a period not longer than six months and at least once in every 50 sites.

2.7 Classification by ground movement

According to the Australian Standard AS2870 (2011), the site classification is based on the expected ground surface movement as shown in Table 3.

Table.3 Site Classification Classes

(AS 2870, 2011)

y_s (mm)	Class	Description
$0 < y_s \leq 20$	S	slightly reactive
$20 < y_s \leq 40$	M	moderately reactive
$40 < y_s \leq 60$	H1	highly reactive
$60 < y_s \leq 75$	H2	highly reactive
$y_s > 75$	E	extremely reactive

Class E site usually contain deep layers of expansive clays and is often associated with a harsh semi-arid climate (i.e. long dry period followed by a short period of relatively high rainfall). A sub-classification is applied based on the depth of the expected moisture change. “-D” is add to the appropriate site Class to indicate that the design depth of suction change (H_s) is greater than 3 m. For example, H1 represents a highly reactive site with shallow moisture change while H1-D represents a highly reactive site with deep moisture change

2.7.1 Factors affecting the ground movement

Numerous factors could affect the soil behaviour, both natural and man-made. The most important reasons are plants, climate and human activities.

2.7.1.1 Plants

Plants contribute to property values by enhancing the aesthetic appearance, screening unsightly views, reducing noise and cutting energy costs. Plants, however, can cause soil desiccation problems. Plants near to a house, especially Australian native species can extract large quantities of moisture from soils leading to localised settlement. If this shrinkage settlement is significant, pavements and residential buildings may deflect significantly and result in structural damage (Li et al 2014).

2.7.1.2 Climate

In nature, most changes in soil water content are caused by seasonal climate variation. Climate affects the depth of soil suction change and the change in suction at the soil surface. In Australia, Thornthwaite Moisture Index (TMI) is used to infer the depth of design seasonal suction change, H_s for the purpose of site reactivity classification. Appendix D of Australian Standard AS2870 (2011) provides a map of Victoria showing five climate zones. TMI is mainly a function of rainfall and potential evapotranspiration. A positive value of TMI indicates a humid or wet climate with higher soil moisture while a negative value of TMI represents an arid climate with less moisture in soil. H_s increases with the harshness of the climate (Li and Sun, 2015). As an example, H_s is 1.8m in the eastern area of Melbourne ($10 > \text{TMI} \geq -5$) but increases to 4m in Adelaide ($-25 \geq \text{TMI} \geq -40$).

2.7.1.3 Human activity

When ground is covered with a residential slab, evaporation of moisture from the soil and rainfall infiltration into the soil is prevented. Assuming that ground water table is deep, then much of the soil moisture exchanges will occur laterally between the edge of footing and the central soil mass. For a slab footing placed on an initially dry site, it will experience an edge heave mode.

Although seasonal variation in climate is a dominant factor of expansive soil volume change, many man made effects such as garden watering, leaking underground water pipes and a deficient storm water drainage system are also responsible for soil volume change.

2.7.2 Ground movement calculation

As mentioned in Section 2.7, site classification is based on y_s (refer to Table 3), the predicted design site surface movement, over the life of the house, which is based on design soil suction change profiles for different climatic regions of Australia. The value of y_s is calculated by the following expression:

$$y_s = \frac{1}{100} \int_0^{H_s} I_{pt} \Delta u \, dh \quad \text{Equation. 10}$$

Where

y_s = characteristic surface movement (mm)

Δu = Soil suction change at depth (z) from the surface, expressed in pF units.

dh = is the thickness of the soil layer

H_s = depth of design suction change

I_{pt} = instability index

Each of the above parameters will be discussed further in the following sections.

2.7.2.1 Suction change at depth (Δu)

The depth of design suction change is the depth from surface to level of the zero suction change (shown in Figure 15). Shallow bedrock is treated as a non-reactive soil and therefore has no effect on the design suction change. Where a permanent water table or shallow bedrock is encountered within the depth of H_s from the surface, the depth of design suction change needs to be modified in accordance with Figure 15. The design suction change at the soil surface is given in Table 4

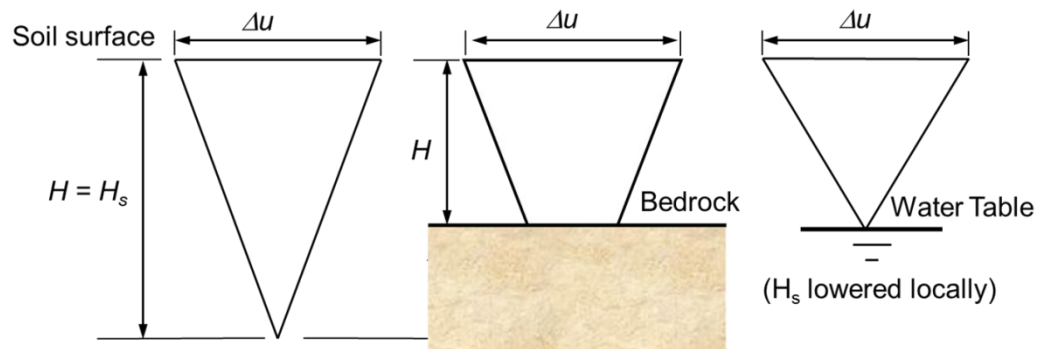


Figure.15 Effect of water table and bedrock on design suction change profiles

2.7.2.2 Depth of site suction (H_s)

The depth of design suction change (H_s) is given in Table 4 for various locations. can be It can be extrapolated to other areas based on climate.

Table.4 Soil suction change profile for certain location in Australia

Soil suction change profiles for certain locations		
Location	change in suction at the soil surface (Δu) pF	Depth of design soil suction change (H_s) m
Adelaide	1.2	4.0
Albury/Wodonga	1.2	3.0
Brisbane/Ipswich	1.2	1.5-2.3
Gosford	1.2	1.5-1.8
Hobart	1.2	2.3-3.0
Hunter Valley	1.2	1.8-3.0
Launceston	1.2	2.3-3.0
Melbourne	1.2	1.8-2.3
Newcastle	1.2	1.5-1.8
Perth	1.2	1.8
Sydney	1.2	1.5-1.8
Toowoomba	1.2	1.8-2.3

2.7.2.3 Instability index

Soil reactivity is classified according to surface ground movement. In order to estimate the surface ground movement we also need to know an important parameter called instability index. The instability index (I_{pt}) is defined as the percent vertical strain per unit change. It could be calculated from the test for the reactivity index (I_{ps}) by using the following equation, provided in the Australian standard:

$$I_{pt} = \alpha \bullet I_{ps} \quad \text{Equation. 11}$$

- In a cracked zone (unrestrained):

$$\alpha = 1.0$$

- In an uncracked zone (restrained laterally by soil and vertically by soil weight):

$$\alpha = 2.0 - z/5 \quad (z = \text{depth from the finished ground level to the point under consideration in the uncracked zone})$$

2.8 Laboratory test for reactivity index

In the AS1289, the reactivity index could be determined by one of the following methods:

- a) Core shrinkage test ([AS 1289.7.1.3](#)).
- b) Loaded shrinkage test ([AS 1289.7.1.2](#));
- c) Shrink-swell test ([AS 1289.7.1.1](#));

Each method has its own advantages and disadvantages. The following sections provide a description of each method.

2.8.1 Core shrinkage index (I_{cs}) test ([AS1289.7.1.3](#))

The core shrinkage test requires an undisturbed soil sample. It monitors the soil shrinkage under air-dry conditions with room temperature controlled at $23 \pm 2^\circ\text{C}$. An undisturbed sample can be obtained from a thin-walled steel tube pushed into the soil. The diameter of the soil sample ranges from 38mm to 65 mm. The length of the sample should not be less than 1.5 times its diameter and not larger than twice its diameter. A drawing pin is placed into the centre of each end of a cylindrical core soil sample to provide a reference mark for monitoring the change in length. During the test, the weight and length of the sample is regularly recorded. After shrinkage ceases, the sample is put into the oven to dry to constant mass at 105° and the final moisture content and length of the sample are measured.

The sample trimming are used to determine the initial soil suction. The soil suction can be measured by WP4 or Wescor HR-33T. Two thin discs of soil 3-4 mm thick are cut from the remaining core sample retained and placed in a vacuum desiccator over a supersaturated solution of ammonium chloride (total suction = 5.5 pF). The mass of the discs is monitored to assess when mass equilibrium is reached. The soil moisture content of the soil discs are measured after mass equilibrium is reached (at least one week), and the corresponding values of soil suction can be assumed to be 5.5 pF.

Knowing the rate of shrinkage strain of the soil sample with change in moisture content the core shrinkage index (I_{cs}) can be calculated using the following equation:

$$I_{cs} = \left| c \times \frac{\Delta \varepsilon}{\Delta w} \right| \times 100 \quad \text{Equation. 12}$$

Where:

$$c = \frac{w_c - w_o}{5.5 - u_o}$$

w_c = the moisture content of the dried soil discs (%)

w_o = the initial moisture content of the sample trimmings (%)

u_o = the initial total suction of the sample

$\Delta\varepsilon$ = the shrinkage strain (%)

Δw = change in moisture content (%)

2.8.2 Load shrinkage index (I_{ls}) test (AS1289.7.1.2)

As outlined by Pile et al. (1984), the load shrinkage test is used to measure the shrinkage of a soil sample under a constant pressure. The test requires a considerably long time (a minimum of 6 weeks) to perform. The soil sample used in the test is also a cylindrical core sample but much smaller than the one used in the core shrinkage test, which has a minimum diameter of 38 mm and a length of 55 mm. A pressure of 25kPa is applied to the sample. The initial suction, moisture content and length of the sample need to be determined before the test. The loaded shrinkage cell is placed in the vacuum desiccator over the saturated copper sulfate solution to allow the soil sample to dry until equilibrium is reached. After removing the sample from the loaded shrinkage cell, the final soil suction and moisture content are measured. The loaded shrinkage index (I_{ls}) can be calculated by the following Equation:

$$I_{ls} = \left| \frac{w_o - w_f}{u_o - u_f} \times S \right| \times 100 \quad \text{Equation. 13}$$

Where:

S = the slop $\Delta\varepsilon/\Delta w$

w_f = the final measured moisture content of the soil (%)

w_o = the initial moisture content of the sample trimmings (%)

u_o = the initial soil suction of the sample trimmings (pF)

u_f = the final soil suction after the sample was removed from the apparatus (pF)

2.8.3 Shrink-swell test (AS 1289.7.1.1)

The shrink-swell test requires two "companion" samples of soil so that a core shrinkage test and a swell test can be conducted. The core shrinkage test in the shrink-swell test takes the form of a simplified core shrinkage test. The process is very similar to the one outlined in section 2.8.1. For the shrinkage test, the sample diameter should be between a minimum of 38 mm and a maximum of 65 mm.

The total shrinkage strain (ϵ_{sh}) can calculate from the following Equation:

$$\epsilon_{sh} = \frac{100 \times (D_0 - D_d)}{H_0} \quad \text{Equation. 14}$$

Where,

ϵ_{sh} = the total shrinkage strain in oven dry conditions (%)

D_0 = the distance between the rounded heads of the pins after their placement (mm)

D_d = the distance between the rounded heads after removal of specimen from oven (mm)

H_0 = the average initial length of the specimen (mm)

In the swell test, the soil sample is installed in a rigid steel ring and placed in a consolidation apparatus. A minimum vertical pressure of 25 kPa is applied and distilled water is usually added to allow soil to swell. For the swell test, the sample should have a minimum diameter of 45 mm.

Knowing the swelling strain, the shrink-swell index can be calculated by the following equation:

$$I_{ss} = \frac{\epsilon_{sh} + \frac{\epsilon_{sw}}{2.0}}{1.8} \quad \text{Equation. 15}$$

Where

I_{ss} = shrink-swell index

ϵ_{sw} = magnitude of the swelling strain. If ϵ_{sw} is < 0 , then assume $\epsilon_{sw} = 0$

ϵ_{sh} = magnitude of the total shrinkage strain in oven dry conditions

In Equation 15 ([AS 1289.7.1.1, 2003](#)), two empirical parameters are introduced, i.e. the lateral coefficient of expansive soil movement of 2 and the soil suction change of 1.8 pF. In this study, a series of tests have been carried out to evaluate these two parameters..

The impact of initial moisture content (suction), sample size and load on the shrink-swell testing are also assessed. The results of these tests will be presented in the following chapters.

2.8.4 Summery of the three test methods for the reactivity index

In the core shrinkage test, the slope of the water content versus suction curve is needed to determine the initial soil suction; the variation in initial suction may result in different core shrinkage index. In addition, the initial moisture state of the soil sample is important if a sample is very dry, no much change in the length of the sample can be recorded.

The load shrinkage test requires the initial soil suction to be measured. Mass equilibration commonly takes 8 weeks or more, which makes this method is more suited to research than consultancy

The shrink-swell test is performed on two "companion" samples from the same core sample. The sample may be either very wet or very dry. Both swell and shrinkage behaviours are considered, and it does not require to measure soil suction. Cameron (1989) found that the shrink-swell test was the most reliable indicator of site reactivity and provided the best indicator of observed ground movement.

2.9 Atterberg limit test

The Atterberg limit test is one of the basic soil tests. It consists of the liquid limits test ([AS1289.3.1.2](#)), the plastic limits test ([AS 1289.2.1.1](#)), and linear shrinkage test ([AS 1289.3.4.1](#)). The Atterberg limit test is used to obtain basic index information of the soil and to estimate its strength and settlement characteristics. This test is named after a Swedish geotechnical engineer, Atterberg who developed the liquid and plastic limits in the early 1900s. Later on, between the 1920s and 1930s; the method was refined and modified to suit geotechnical works by Arthur Casagrande and Terzaghi.

As the soil water content changes from oven dry to saturation, it progresses through different phases: brittle solid, semi-solid, plastic and liquid (Figure 16) The Atterberg limits, such as the liquid limit or plastic limit, are the boundary lines between two states.

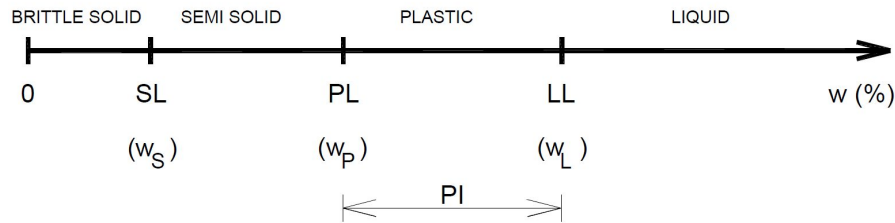


Figure.16 Relationship in Atterberg limit

2.9.1 Liquid limit

The liquid limit test aims to empirically establish the moisture content at which soil starts to behave as a liquid material. Two main types of test are specified. The first is the cone penetrometer method, which is fundamentally more satisfactory than the alternative because it is essentially a static test depending on soil shears strength. It is also easier to perform and gives more reproducible results. The second is the Casagrande type of test, which has been used for many years as a basis for soil classification and correlation of engineering properties. In this research project, the Casagrande method is used to determine the liquid limit.

The formula for calculating the liquid limits show in Equation 16: (AS 1289.3.1.2—2009)

$$w_L = w \cdot \left(\frac{n}{25}\right)^{\tan \beta} \quad \text{Equation. 16}$$

Where:

w_L = liquid limit, in percent

w = moisture content in percent, corresponding to n blows

n = number of blows to closure

$\tan \beta = 0.091$ for a range of Australian soils

2.9.2 Plastic limit test

The plastic limit of a soil is a measurement of the moisture content at which the soil changes from the plastic condition to a semi-rigid, friable state. It is the moisture content at which a soil will just begin to crumble when rolled into a 3 mm diameter using a ground glass plate. The test is performed according to the AS 1289.3.2.1 – 2009.

In order to calculate the plastic limit w_p , the formula is taken from another Australian Standard, AS1289.2.1.1- 2005, as shown below:

$$w_p = \frac{m_b - m_c}{m_c - m_a} \quad \text{Equation. 17}$$

Where:

w_p = moisture content for plastic limit, in percent

m_b = mass of container and wet soil, in grams

m_c = mass of container and dry soil, in grams

m_a = mass of container, in grams

2.9.3 Plasticity index of soil

When both liquid limit and plastic limit are determined, the difference between the two limits is called the plastic index:

$$I_P = W_L - W_P \quad \text{Equation. 18}$$

Where

= plasticity index, in percent

= liquid limit, in percent

= plastic limit, in percent

2.10 Linear shrinkage test (AS 1289.3.4.1)

The linear shrinkage measures the soil's horizontal shrinkage at its liquid limit. It uses the same sample as the liquid limit test, which is placed in a shrinkage mould and air dried followed by oven drying at 105°C for 24 hours. Then, the longitudinal shrinkage is measured.

The percentage of linear shrinkage of the specimen is calculated using the following equation (AS 1289.3.4.1, 2008):

$$LS = \frac{L_s}{L} \times 100\% \quad \text{Equation. 19}$$

Where:

LS = linear shrinkage of the sample, in percent

L_s = longitudinal shrinkage of the specimen in millimeters

L = length of the module in millimeters

2.11 Summary

This chapter has briefly introduced the research project and related laboratory tests, and explained the classification of expansive or reactive soil sites for residential construction. The design surface movement is calculated based on the design suction changes given in the Australian Standard AS2870 (2011) and the reactivity or instability index. The instability index can be determined from three methods: core shrinkage test, loaded shrinkage test and shrink-swelling test. The shrink-swelling test is the most reliable indicator of site reactivity and provides the best indicator of observed ground movement. An obvious advantage of the shrink-swell test is that the initial moisture state of the sample is not important. The sample can be either very dry or very wet. Other advantages are (1) it does not require measurement of soil suction and (2) it is relatively quick.

In subsequent chapter, the laboratory tests will be described in details. The testing results will be presented and discussed.

CHAPTER 3: LABORATORY TESTS

3.1 Introduction

This research is mainly based on experimental work conducted in the geotechnical laboratory of the School of Civil; Environmental & Chemical Engineering at RMIT. In this chapter, the types of testing, the methodology and procedure of experimental work are described. The sample collection and preparation are also described, which include the description of soils tested, locations, depths, physical properties and experimental setup.

The laboratory tests are generally divided into two parts. The first part consists of the standard tests, which follow the recommendations of the relevant Australian Standards. The results of these tests were used to compile a small library of geological information about the Melbourne metropolitan area. In the second part of the laboratory tests, a series of comparative and verification tests were conducted. The objective was to evaluate the laboratory tests for reactivity of expansive soils.

All the testing was conducted in accordance with the Australian Standards. The following Australian standards were used for this research:

- AS 1289.7.1.1 – 1998: Soil reactivity tests - Determination of the shrinkage index of a soil – Shrink- swell index
- AS 1289.3.1.2 – 2009 Soil classification test – Determination of the liquid limit of a soil
- AS 1289.3.2.1 – 2009 Soil classification test – Determination of the plastic limits of a soil – Standard method
- AS 1289.3.3.1 – 2009 Soil classification tests – Calculation of the plasticity index of a soil
- As 1289.3.4.1 – 2008 Soil classification tests – Determination of the linear shrinkage of a soil – standard method

3.2 Sample location

One of the main objectives of this study was to build up a database of shrink-swell indices of soil in the metropolitan area of Melbourne. This means that a large amount of soil samples that cover a range of soil, geological and geographic conditions are required.

As a result, 60 undisturbed samples were collected from 47 different field sites across Victoria area for the laboratory testing. Most of the samples were collected/provided by RMIT University's industry partner, FMG Engineering. An interactive map, as shown in [Figure.17](#), has been generated based on the Google map. This map shows the location of the sites where soil samples were collected and the detailed information about the test results.

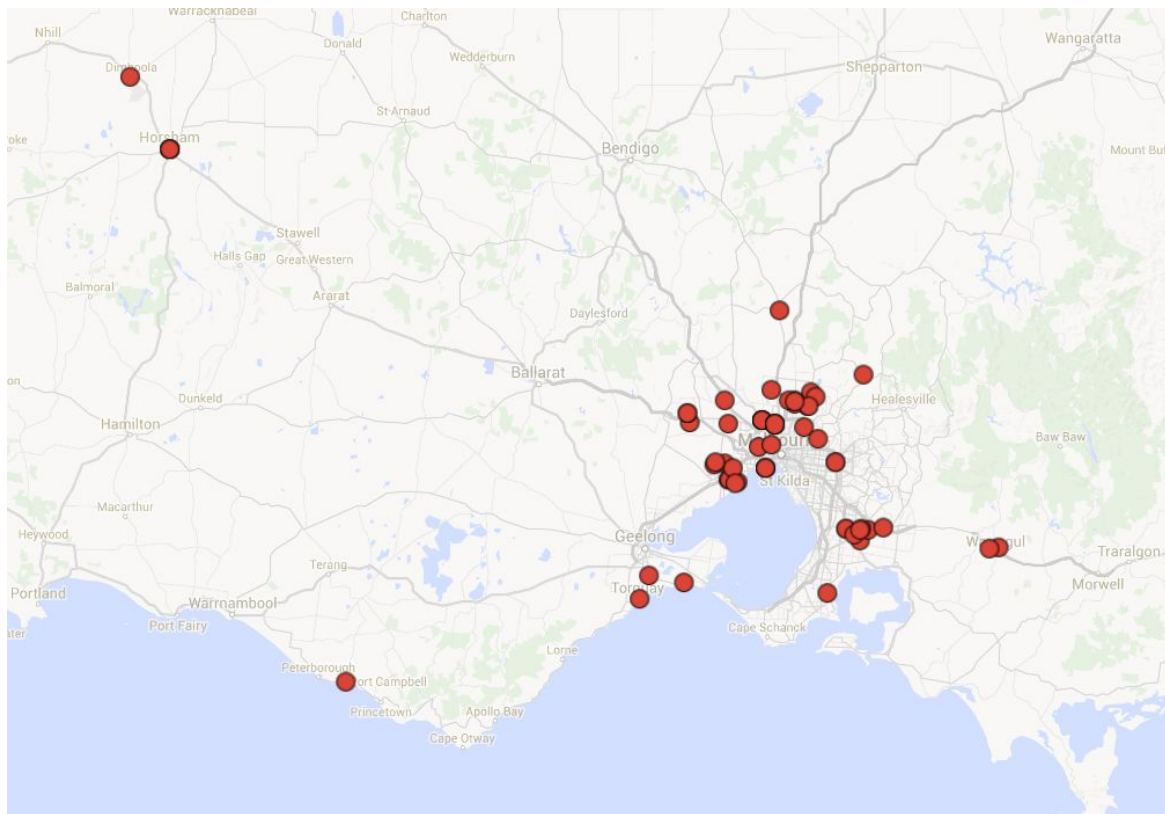


Figure.17 The location map showing sites where soil samples were collected in Victoria

The undisturbed soil samples were taken by driving thin walled stainless steel tubes (450 mm in length and 50 mm or 60 mm in nominal inner diameter) into the ground. The hydraulic drilling rig used for sample collection is shown in [Figure 18](#). The steel tube with sharp edges was fastened to an extension rod by using an adaptor, and the rod was affixed to the hydraulic

hammer and lowered into position, a hydraulic mechanical hammer provided enough power to push the tube into the soil. The sampling tube was then removed from the borehole, and the ends of the tube were immediately sealed with tape to minimise the loss of moisture. The tube was then sealed into an airtight plastic bag with soil information.



Figure.18 Sample collection from field sites

3.3 Laboratory tests

Laboratory testing is an essential component of this study, as the information about soil properties and the research data are all derived from laboratory tests. RMIT Geotechnical Laboratory provides all the equipment and tools required for this research.

3.3.1 Shrink-swell testing – sample preparation

The steel tubes containing soil samples were sealed and labelled immediately on extraction from the borehole on site and then transported to the laboratory. Prior to testing, the sample was extracted from the tube with either a hand operated jack or semiautomatic continuous hydraulic jack (Figure 19) and then cut and trimmed carefully using a cut ring and a sharp spatula to suitable size (Figure 20).

The shrink-swell test consists of a core shrinkage test and a swelling test. It requires two identical soil samples which have the same initial moisture content. The two soil samples (one for core shrinkage test and another for swelling test) come from the same tube/core sample.

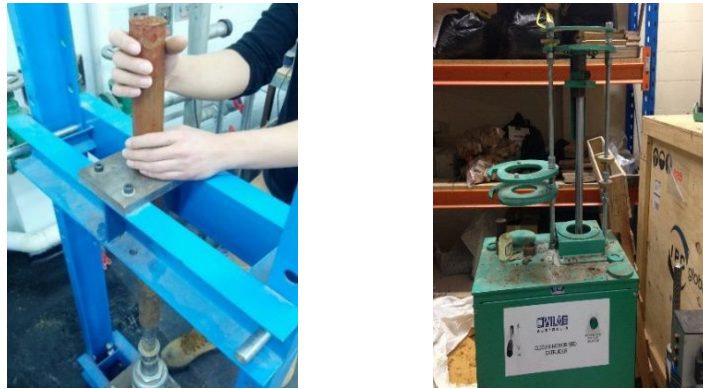


Figure.19 Equipment used to extrude soil sample out of steel tube



Figure.20 Preparation of soil samples for shrink swell tests

3.3.1.1 Shrink swell testing - core shrinkage test

The core shrinkage test requires an undisturbed cylindrical core sample of a diameter of 38-65 mm to be cut / trimmed to a length within the range of 1.5 to 2 diameters (Figure 21). Drawing pins are placed in the center of the core at either end to provide a reference mark for measurements to be taken. The specimen is allowed to be air dried for about 1-2 weeks with mass and length measurements taken throughout this period. After about two week drying in the air, the core is then oven dried about 24 hours to a constant mass. When the sample is removed out of oven, a measurement of the distance between the drawing pins is taken and final moisture content is determined. The total shrinkage strain ϵ_{sh} is then calculated using the equation 14:

$$\varepsilon_{sh} = \frac{100 \times (D_0 - D_d)}{H_0} \quad \text{Equation 21}$$

Where

- D_0 is the distance between the rounded heads of the pins after their placement (mm);
- D_d is the distance between the rounded heads of the pins after removal of specimen from oven (mm)
- H_0 is the initial length of the specimen (mm)

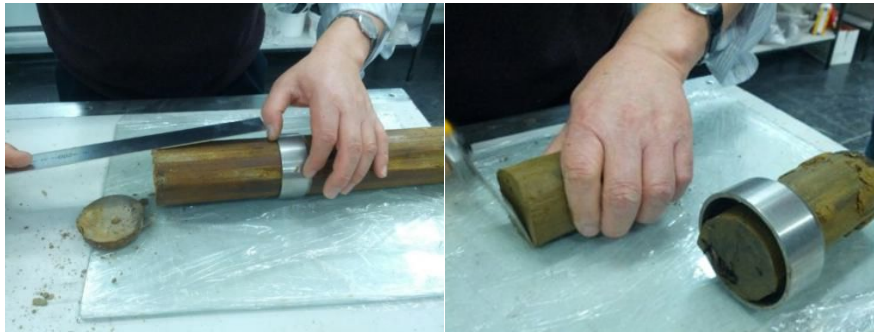


Figure.21 Specimens for core shrinkage test shall be core cylinders having a length to diameter ratio of 1.5 -2.0.

After cutting and trimming, the specimen is placed on a smooth surface, allowing it to dry out for a period of at least 10 days. The specimen is wrapped with plastic membrane overnight to alleviate moisture gradients within the specimen and reduce the risk of cracking. In this study, the changes in specimen length, diameter and weight were monitored twice a day (Figure 22). The distance between the rounded heads of the pins was measured and recorded with a digit vernier calliper to the nearest 0.01mm. When the change in length was less than 0.1 mm over the last 24 hours, the specimen was placed into the oven and dried to a constant mass at 105 °C degrees. Its final moisture content was then registered, as well as its final length. After having these data, a plot of strain vs. moisture content could be generated, and the specimen was used to determine the Atterberg limits.

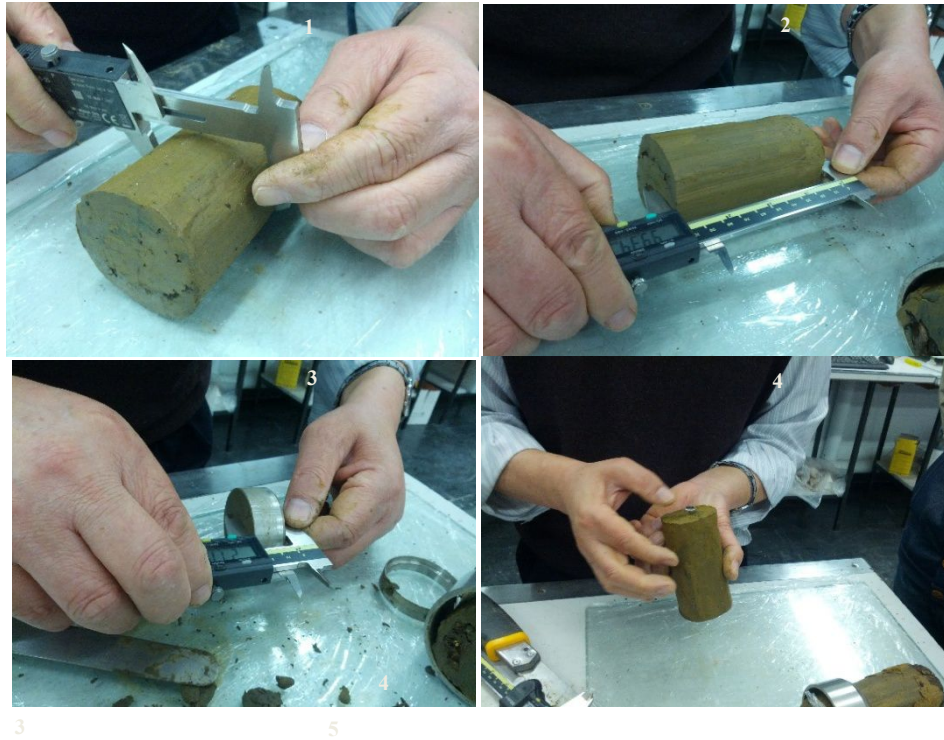


Figure.22 Recording of changes in length of specimen during core shrinkage test

3.3.1.2 Shrink swell testing -swelling test

For the swelling test, a 50 mm diameter sample was cut with a rigid steel ring of 20 mm height and 45 mm in diameter and trimmed carefully to ensure both ends were flat ([Figures 23 and 24](#)). The trimmings were collected and used to determine initial water content and suction. The specimen was then placed in a consolidation cell with two porous stone plates at the top and bottom. A seating pressure of 5 kPa was initially applied for about 10 minutes and the dial gauge/displacement transducer was zeroed under this seating load to allow for a small amount of initial settlement of specimen. The vertical pressure was then increased to a value equal to the overburden pressure or 25 kPa (whichever is greater) for a maximum period of 30 minutes. After recording the initial specimen settlement, which was taken as the datum from which swelling strain was determined, the specimen was inundated with distilled water and allowed to swell. The testing was continued until the swelling increment was less than 5% of the total specimen swelling movement for a period of at least of 3 hours.

To prevent the soil from swelling out of the steel ring during the swelling test, a 5 mm extension ring with the same internal diameter was installed to the top of swelling ring to

allow for the possible expansion of soil above the consolidation ring.



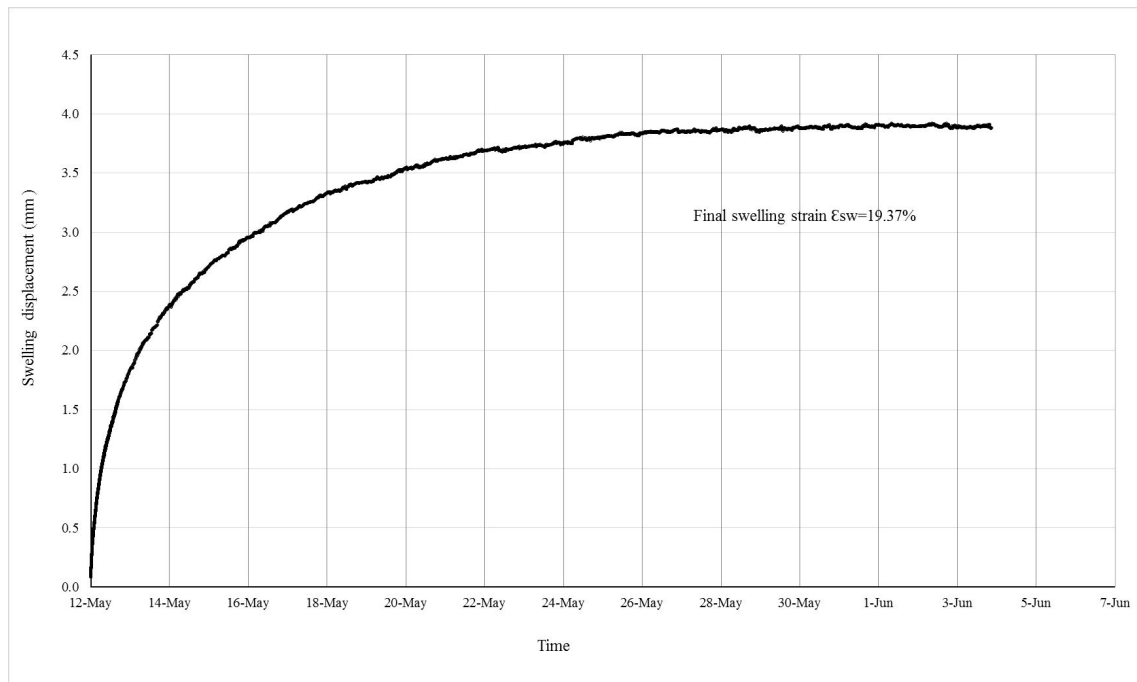
Figure.23 Sample preparations for swelling testing



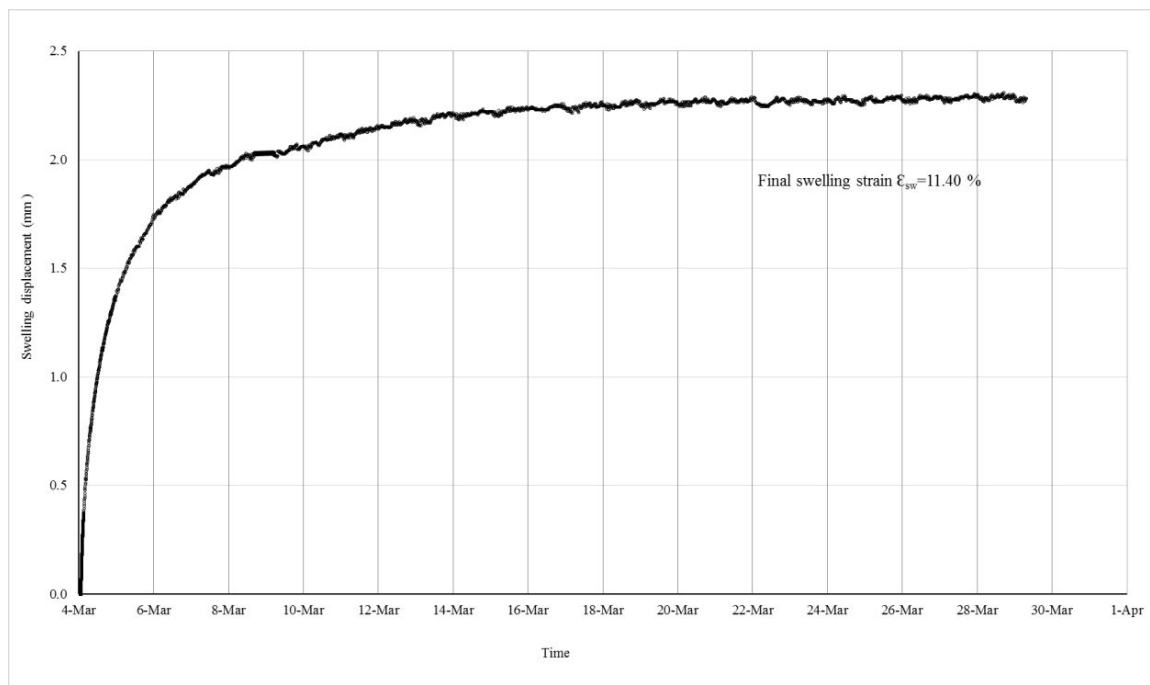
Figure.24 Apparatus used in the swelling tests

In this study, the swelling specimen was allowed to heave for a period of at least of 12 days. The typical results of swelling tests are presented in [Figure 25](#). It is interesting to note that only 60% of total swelling strain of the Williams landing specimen was completed in the first 48 hours while 74% of total swelling strain for the South Morang specimen. The results show that for highly reactive clay it may need more than 10 days to finish a swelling test.

After the swelling test, the specimen was extracted from the ring and the final water content and suction were determined.



(a) Soil sample from Williams Landing, Vic.



(b) Soil sample from South Morang, Vic.

Figure.25 The measured heave of the specimens *versus* time

The total swelling strain ε_{sw} is calculated by using the following equation:

$$\varepsilon_{sw} = \frac{100 \times (D_s - D_i)}{H_0} \quad \text{Equation. 22}$$

Where ;

D_s is the total swell of the sample after inundation. (mm);

D_i is the initial settlement observed prior to inundation (mm) ;

H_0 is the average initial height of the specimen (mm).

3.3.2 Atterberg limit tests

Atterberg limit tests were also conducted in this study, in which soil samples were prepared in accordance with AS 1289.1.1. [Figure 26](#) shows the apparatus used in the testing.



Figure.26 Apparatus used for Atterberg limit tests

3.3.2.1 Preparation of soil for Atterberg limits

The soil sample used in Atterberg limit test was obtained from the core shrinkage test. After the shrinkage test, the sample was dried in the oven for at least 24 hours and crushed with a hammer, to break it into small particles and remove stones and roots. (Figure. 27)

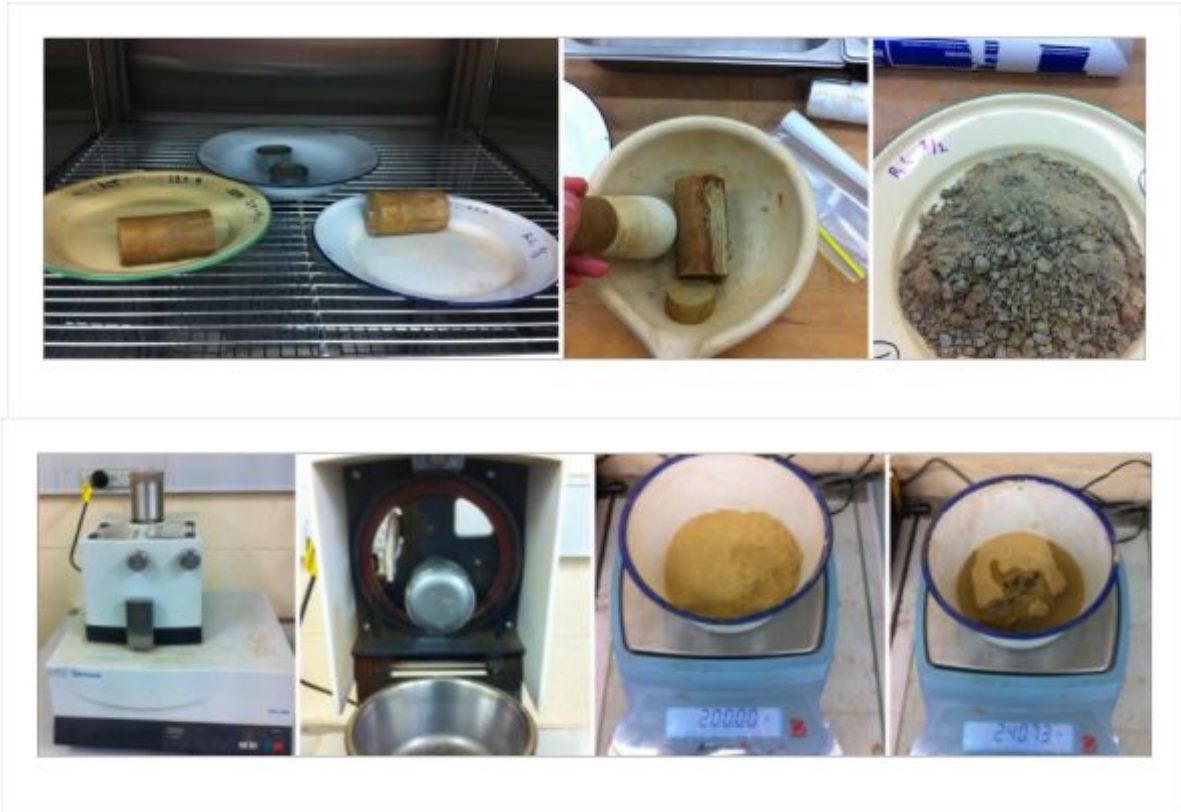


Figure.27 Preparation of soil samples for Atterberg limits test

The soil sample was then placed in the mortar-grinder machine and ground into smaller particles.

The Australian standard requires the soil sample to be smaller than 425 μ m, so the soil powder was sieved using 425 μ m sieve.

3.3.2.2 Liquid limit test

The liquid limit test requires about 300 gram of soil sample with particle size smaller than 0.425mm. Distilled water was sprayed and mixed with dry soil until the sample reached the consistency of a paste. Before the test, the soil needs to be mixed with distill water and to cure in a sealed container for at least 24 hours ([Figure.28](#)).



Figure.28 Preparation of the sample for the liquid Limits Test

The drop height of the Casagrande cup was calibrated using a spacer gauge to ensure that height to which the cup was lifted was exactly 10 mm above the base. The soil sample was then placed in the cup by using a spatula. The soil needs to be carefully leveled off at the top ensure that no air is entrapped. ([Figure 29](#))

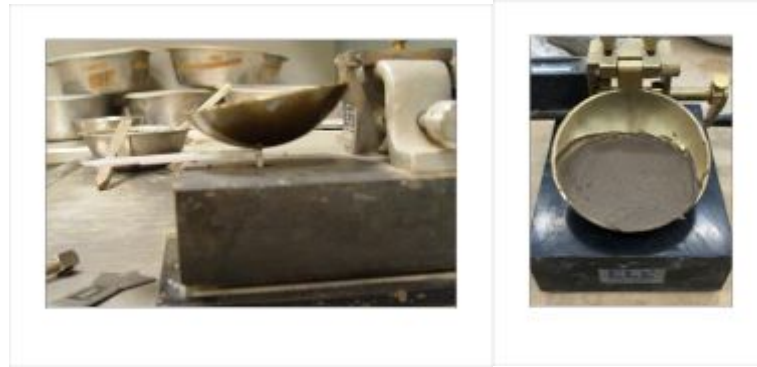


Figure.29 Checking the cup height and levelling off at the top ensuring no entrapped air

The thickness of the soil cake in the apparatus was about 10 mm height. A brass grooving tool was used to cut the sample into two parts. The groove was in the middle of the soil cup and the distance between the two soil cakes was about 2mm. (Figure.30)



Figure.30 Casagrande device and grooving tool

During the test, the crank handle of the apparatus should be kept at a constant speed of 2 rev / sec. The number of blows that cause the closure of the groove was counted. The standard requires the closure of the groove to occur at 25 blows; if closure occurs at a higher or lower number of blows, the moisture of the soil sample needs to be adjusted by the operator so that it takes about 25 blows for the groove to touch.

After closure was successfully achieved at 25 blows, the soil samples were collected to determine the moisture content, which is known as the liquid limits for the soil sample.

3.3.2.3 Plastic limit test

The plastic limit test takes about 30 grams of dry soil sample from prepared soil. For the same purpose as the liquid limit test preparation, the soil samples need to be mixed with distilled water and left it in a sealed container to cure for 24 hours. The sample should not be too wet or too dry, just enough to make the sample into a ball as show in [Figure 31](#).

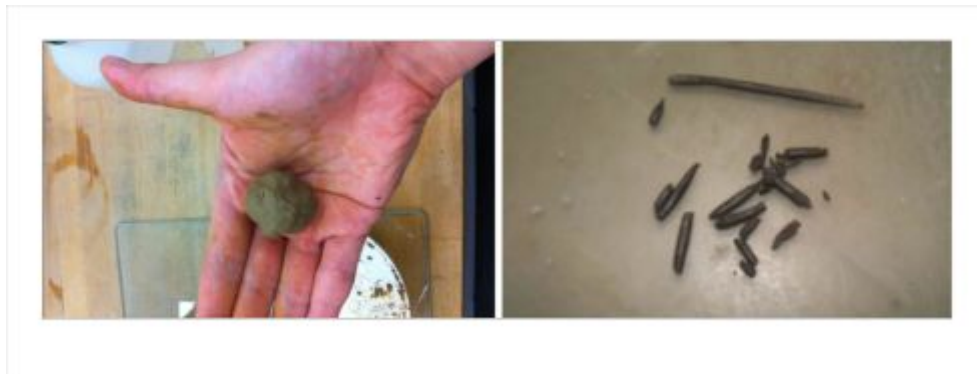


Figure.31 Plastic limit test

Taking out 6 grams of the prepared soil and rolled between the operator's palms or on a glass plate. The rolling speed should be between 80 to 90 strokes per minutes and the soil should form a 3 mm diameter thread; if the soil crumbles while forming a thread of about that size (3mm), the pieces of crumbled soil thread can be collected into a tin and used to determine the moisture content. ([Figure 31](#)) On the other hand, if the thread reaches a diameter of less than 3 mm without any cracks appearing (which means the soil sample is too wet), the soil moisture is higher than the plastic limit. In this case, the soil should be air dried for a while and the process is repeated until there is enough soil to have attained the test requirement.

As 1289.3.2.1 – 2011 requires that a second test is needed and the moisture content for both needs to be within 2% of each other; otherwise, the test is considered to have failed and needs to be repeated until this criteria is attained. All the crumbled samples are then collected and weight in the moisture tin for water content determination, which is the plastic limit of the soil.

3.3.3 Linear shrinkage test

The linear shrinkage mould used in the test is half a cylinder with an edge in each ends. The shrinkage mould is 140mm long, has a diameter of 12.5mm and is 20mm high from the top to the bottom edges. (Figure 32)



Figure.32 Linear shrinkage test sample

A thin layer of Vaseline is smeared to prevent the soil from sticking to the mould, and to prevent air from mixing with soil in the mould, the mould was shaken for a few times after being filled with soil. The vibrations make the air bubbles and free water come out of the soil. Then, the top layer was scraped and the top edge was smoothed. (Figure 33)



Figure.33 Linear Shrinkage test (before oven dry)

The shrinkage mould is left in the air for about 24 hours, and then it is dried in an oven at 105 degrees centigrade. The length of the dry clay is measured and these results are used to calculate the linear shrinkage strains. (Figure 34)



Figure.34 Linear Shrinkage test (after oven dry)

3.4 Evaluation of the Standard method -AS1289.7.1.1 for calculation of shrink-swell index.

A number of assumptions were introduced in AS1289.7.1.1 (2003) in order to simplify the testing and calculation of shrink-swell index. For example, to avoid the requirement of measurement of soil suction, the range of soil suction change corresponding to the soil volume change is assumed to be equal to 1.8 pF units for all soils. This assumption is based on the collective experience of the AS2870 code committee. The second significant

assumption of the shrink swell test relates to the use of the factor of 2.0 to estimate the vertical swelling strain in an unconfined swell specimen, from the measured vertical strain in a rigidly confined swell specimen (Fityus et al, 2005).

In this research, a comprehensive experimental study was carried out to evaluate the assumptions introduced in AS1289.7.1.1 (2011)

3.4.1 Range of soil suction change

To evaluate the range of suction change, the soil suction measurement was taken not only before each soil shrink-swell test (i.e. initial soil suction), but also after swelling test and core shrinkage test. Total suction was measured by using a WP4 Dewpoint Hygrometer (range: 0 to -300 MPa with an accuracy of 0.1 MPa) and a Wescor Dew Point Microvoltmeter HR-33T (range: -0.1 to -8 MPa).

3.4.2 The correction factor for shrinkage test

The second problem is a theoretical problem. In the shrink-swell test, the axial swell strain is obtained from one-dimensional consolidation test while the axial shrinkage strain is measured from an unrestrained core shrinkage test. This discrepancy is corrected by dividing the swelling strain measured in the restrained test by a factor of 2.0.

It is very hard to conduct a three-dimensional swelling test to assess the assumed lateral restraint factor of 2. In this study, unrestrained core shrinkage tests were performed on Maryland clay (reconstituted sample consolidated from slurry) and Braybrook clay (compacted remoulded samples). During the testing, the change in length, diameter and volume of samples were closely monitored by using an Artec Spider 3D scanner and a digital vernier caliper. The volumetric strain was recorded and compared with the measured axial strain and radial strain.

3.4.3 Issue of soil sample swelling out of the sample ring

Fityus (1996) found that highly reactive soil might swell out of the steel ring during the swelling test, leading to an inaccurate result in the swell test. To overcome this problem,

Fityus (1996) suggested adding an extension annulus on top of the soil sample to allow for the possible expansion of soil above the consolidation ring. However, for extremely reactive clay samples, a loss of soil still occurred even if an extension ring was adopted. In a number of swelling tests, it was observed that the extension ring was lifted from the top edge of the oedometer ring by highly reactive soil samples. Soil leaking from the bottom of the consolidation ring was also observed in some cases. In order to prevent the soil from swelling out of the oedometer ring, a few different methods had been tried in this study. It was decided to add two extension rings, one at top and one at bottom. The height of the bottom ring is the same as the thickness of the bottom porous stones so the height of the specimen was kept at about 20 mm. Before swelling testing, the extension rings were tied up with a hose clamp to stop them from moving (refer to Figure 35).



Figure.35 Swell test samples after swell test

In order to prevent the soil from swelling out of the oedometer ring, a few different methods had been tried in this study. It was decided to add two extension rings, one at top and one at bottom. The height of rings was the same as the thickness of the top and bottom porous stones so the height of the specimen was kept at about 20 mm. Before putting the specimen into the consolidation ring, the extension rings was tied up with a hose clamp to stop them from moving (refer to Figure 36).



Figure.36 Swelling sample with fixed extension ring

3.4.3 Suction measurement

3.4.3.1 Suction measured by using Wescor HR-33T (AS1289.2.2.1)

Compared to WP4, it is more difficult to operate Wescor HR-33T. The basic operational procedures are as flows (AS1289.2.2.1, 1998):

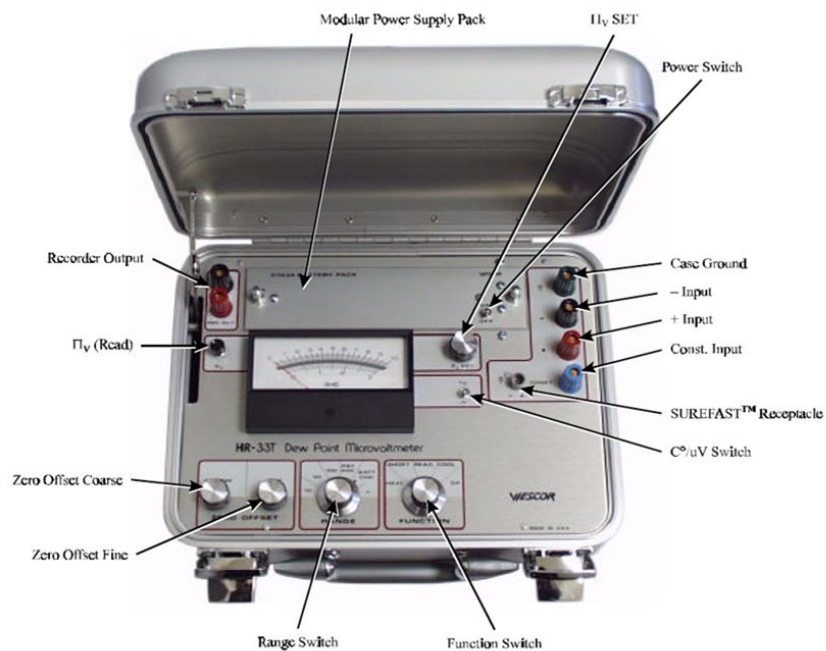


Figure.37 Wescor's Dew Point Microvoltmeter, model HR-33T

- (a) Inspect the terminals on the microvoltmeter and check that the chamber wire connections are tight and correct. Record the chamber number.
- (b) Check the temperature in the chamber using the temperature switch on the microvoltmeter. Record the chamber temperature.

- (c) Set the cooling coefficient (π_v) of the chamber as described in the manufacturer's instruction manual, or adjust π_v for the chamber temperature based on past recorded variation of p_v with temperature. Record the value of π_v . Switch off the microvoltmeter.
- (d) From the soil sample, obtain a soil subsample of minimum diameter 7 mm and cut a 3 to 4 mm thick slice of soil to fit the standard sample cup, 9.5 mm diameter and 4.5 mm deep. Avoid contact of the subsample with hands.
- (e) Quickly transfer the subsample to the sample cup and trim the soil surface to ensure that the soil will not contact the thermocouple junction when placed inside the sample chamber. Clean the surface of the sample cup with a lint free paper towel or tissue, such as Kimtech Kimwipes. Place the cup in the sample chamber and seal the sample chamber. Record the time.
- (f) Wait for the required equilibrium time (30 – 90 seconds depending on soil suction) and then read the soil suction in microvolts as follows:
- (i) Turn the microvolt meter ON with the FUNCTION switch on SHORT, adjust the reading to zero, then wait approximately 30 seconds.
 - (ii) Turn the FUNCTION switch to READ. If the difference between SHORT and READ readings is greater than 2 μV , temperature equilibration is not satisfactory and further equilibrium time is required. Repeat procedure until equilibrium is apparent.
 - (iii) Check the temperature in the chamber and record it. Adjust π_v accordingly.
 - (iv) Select an appropriate output voltage range and set the voltmeter to zero.
 - (v) Set the FUNCTION switch to COOL for the required cooling time (5-25 seconds depending on the soil suction).
 - (vi) Set the FUNCTION switch to DEW POINT and record the voltage output in microvolts to the nearest 0.1 at the observed stable plateau point.
 - (vii) Calculate the soil suction using the calibration equation for the sample chamber. If the equilibrium time for the calculated suction is not satisfactory, turn the FUNCTION switch to HEAT for 3 seconds, then to SHORT. Switch the microvolt meter off. Wait a minimum of 10 minutes and repeat Step (f).
- If the reading seems unusually low, or if the output of the thermocouple junction takes longer than normal to return to zero after heating, the thermocouple may be contaminated. Clean and dry the thermocouple in accordance with the manufacturer's instruction manual and check the calibration. The test may then be continued using a new subsample.
- (g) Repeat Steps (d) to (f) on a second subsample.

(h) If the two readings provide suctions which differ by either more than 0.2 pF for suctions less than or equal to 3.6 pF, or more than 0.1 pF for suctions greater than 3.6 pF, an additional subsample needs be tested.

3.4.3.2 Calibration process

In order to successfully perform the calibration testing and analysis for suction, it is first necessary to prepare sample solutions using Sodium Chloride – pure NaCl – Salt and pure distilled water.

Both the sodium chloride and the pure distilled water were obtained from the Chemical Engineering Laboratory at RMIT. Along with these products, a spatula with an accurate tip was also obtained in order to accurately add any extra NaCl needed to precisely make up the required amount of salt to mix with each solution of distilled water – as a couple of grains can make the weight accurate. The solutions prepared were and the respective amounts of NaCl that had to be mixed are given in Table 5. A digital analytical electronic balance with accuracy to 0.0001 g was used for measuring the salt amounts to be mixed and also the amount of water.



(1)HR-33T Dew Point Microvoltmeter



(2) Psychrometer calibration in salt solution

Figure.38 Calibration with Sodium Chloride solutions

Table.5 Solutions prepared for calibration

Solutions prepared (pF)	Amounts of NaCl (gms NaCl/kg water)
3.0	1.2226
3.5	3.9336
4.0	12.6680
4.5	40.008

Small filter papers provided by the manufacturer were used in the calibration. The filter papers were first soaked into the solution of NaCl and water. Before placed in the cell, they were shaken off for any excess solution – usually about 1 or 2 big drops would come off.

Once the readings have all been taken, there is a set of formula used for the calculations for turning the microvolt readings into meaningful suction values. The formula used in this study is as follows:

$$u = (\log_{10} \text{reading in microvolts}) \times A + B$$

where A and B are constants for each chamber determined by calibration.

3.4.3.2 Suction measured by using WP4

WP4 Dewpoint Hygrometer ([Figure 39](#)) is easy to operate, and the measurement is relatively fast and simple. The soil sample is put in a plastic or steel cup with a diameter of 4 cm and a height of 1 cm. It has a capacity of 15 ml. The sample cup should not be filled more than half full to ensure that the sensor is not in contact with soil. Overfilled cup may contaminate the sensors in the chamber. After a waiting period of 5-10 minutes, the relative humidity in the soil equilibrates with the relative humidity in the air at the chamber. The apparatus will measure the relative humidity and automatically calculate the suction of the soil based on the psychometric law.



Figure.39 WP4-T used for soil suction measurement

3.5 Preparation of remoulded soil samples

There are a number of factors that may have an impact on the values of the instability indices obtained from the shrink-swell test. In this study, a series of shrink-swell tests were performed to evaluate the effects of the initial soil moisture (suction), sample size and loading pressure on the shrink-swell test results. To avoid possible effects due to anisotropy and non-homogeneity within soils, remoulded soil samples were used in the experiments. The sample preparation procedure is described below.

To eliminate possible effects due to natural differences between the composition and structure of undisturbed samples, as well as anisotropy and non-homogeneity within individual samples, the samples used in the experiments were remoulded.

The disturbed soil collected from the field site was firstly dried in an oven at a

temperature of about 100°C until it became non plastic and was then crushed manually by a hammer after removing the roots and small stones. The crushed soil was sieved using a 2.36 mm sieve to remove all coarse materials and vegetable matter. The soil passing through the sieve was then collected, put into a grinder machine (Figure 40) and ground into fine particles. The soil was prepared in a single batch with sufficient quantity to produce all of the required samples.



Figure.40 Preparation of Disturbed Soil Samples for Testing

Four different diameters (i.e. 38mm, 45mm, 50mm and 60mm) of samples were used in this study to evaluate the effects of sample size on the shrink-swelling tests. In routine practice, 45mm or 50 mm diameter samples are widely used while 38mm and 60mm diameter samples are rarely used in shrink-swell tests. Steel rings with different diameters were used to prepare these samples so that all four soil samples had the same initial moisture content and dry density.

Three groups of samples with different initial moisture (suction) were prepared and used for evaluating the effect of different initial moisture on the shrink-swelling tests.

In order to keep the same dry density and void ratio for each sample, the soil sample was compacted by using a tamping rod under the same pressure (refer to Figure 41).

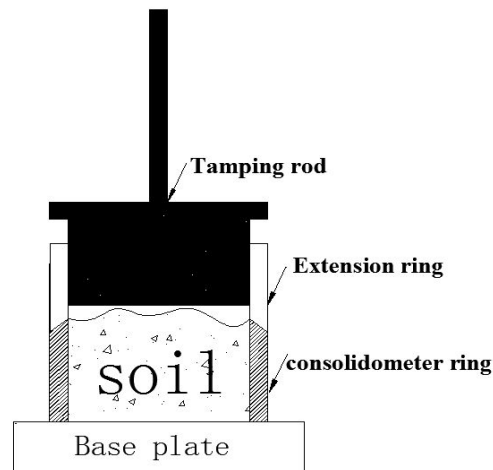


Figure.41 Prepare the soil sample by using tamping rod

3.6 Summery

This chapter has described and discussed the types of testing conducted in this research, sample preparation, the methodology and procedure of experimental work. The testing results will be presented and discussed in the following chapter.

CHAPTER 4: SUMMARY OF TESTING RESULTS AND DATA ANALYSIS

4.1 Introduction

In this research, a series of laboratory tests have been carried out for assessment of the reactivity of expansive soils in Melbourne Metropolitan Area, which include shrink-swell test, Atterberg limits tests, linear shrinkage, particle size distributions, soil suction measurement, X-ray diffraction (XRD) test, and scanning electron microscope (SEM). The test procedure and methodologies were described in Chapter 3. This chapter summarises and analyses the testing results.

First, the results of the shrink-swell tests are used to create an interactive map using Google Maps API which provides information about the nature and distribution of expansive soils in Melbourne metropolitan area. This map/dataset details the location and depth of soil samples collected, site geology information, soil index properties and shrink-swelling indices (I_{ss}).

Secondly, the method/equation for calculation of shrink swell index given in AS1289.7.1.1 (2003) is reviewed. The appropriateness of using a suction range of 1.8 pF and a lateral restraint factor of 2 for all soils is assessed.

In addition to shrink-swell tests, other laboratory tests such as the liquid limit, plastic limit and linear shrinkage etc are also performed in this study. The summary of index properties of soil samples and their correlation with shrink swell indices are presented in this chapter.

Finally, the impact of initial soil suction, sample size and surcharge loading on the shrink-swell test are also discussed.

4.2 Summary of the results of shrink-swell tests

The shrink-swell test, described in AS1289.7.1.1 (2003), is the most commonly used laboratory test for assessing reactivity of expansive soils and site classification in Australian geotechnical practice. Compared to the core shrinkage test and loaded shrinkage test, the

main advantages of the shrink-swell test are (1) both swell and shrinkage strains are taken into account and so, the initial moisture state of the sample may not be important, (2) the measurement of soil suction is not required, and (3) the method is relatively quick, requiring 1-2 weeks to complete. Therefore the shrink-swell test has been chosen in this study to obtain reactivity indices of Melbourne soils.

4.2.1 Summary table of shrink-swelling indices

In this study, a total of 71 samples were collected from 47 different sites in 37 suburbs across Melbourne metropolitan area for the shrink swell tests. These samples were obtained from soils of varying geological origin. However 11 samples were abandoned as they could be extruded out of the damaged steel tubes.

The results of a total of 60 shrink-swelling tests are summarised in [Table 6](#). There is a significant variation in values of the shrink-swell index, I_{ss} , which vary from 0.34 to 10.80 %/pF with a mean of 3.78 %/pF. The results indicate that the western and northern suburbs of Melbourne are covered with highly reactive soils. The soil samples from Williams Landing, Truganina, Tarneit, Wollert, Diggers Rest and Wollert have a shrink-swell index higher than 6%/pF while Dimboola, 330 km north-west of Melbourne, has the lowest shrink swell index of 0.34 %/pF.

Table.6 Summary of shrink-swelling test results

NO .	Location	Depth (m)	Soil types	ϵ_{sw} (%)	ϵ_{sh} (%)	I_{ss} (%/p F)
1	Armstrong Creek	0.75	Quaternary Basaltic Residual soil	0.07	9.05	5.05
2	Ascot vale	0.50	Quaternary Colluvium	0.00	2.00	1.11
3	Braybrook	0.75	Quaternary Basaltic deposits	3.06	6.63	4.53
4	Brookfield	0.60	Quaternary Basalt	1.42	1.93	1.47
5	Clyde north	0.75	Tertiary Basaltic Residual Soils	0.00	3.37	1.87
6	Clyde north	0.95	Quaternary Swamp and Lagoon Deposits	2.91	6.13	4.21
7	Clyde north	1.50	Tertiary Sedimentary	0.00	6.30	3.50
8	Craigieburn	0.90	Quaternary Basaltic Clays	5.43	3.68	3.55
9	Cranbourne north	0.75	Quaternary Swamp and Lagoon Deposits	0.00	4.36	2.42
10	Cranbourne North	0.65	Tertiary Aged Sedimentary deposits	1.54	3.03	2.11
11	Diggers rest	0.65	Quaternary Aged Basaltic Residual Soils	2.42	10.98	6.77
12	Dimboola	1.20	Quaternary aged fluvial deposits of the Coonambidgal Formation	0.00	0.61	0.34
13	Doreen	0.40	Silurian Sedimentary	4.18	5.15	4.02
14	Epping	0.40	Quaternary Basaltic deposits	7.97	3.75	4.30
15	Glenroy -1	0.85	Quaternary Basaltic deposits	3.25	6.30	4.40
16	Glenroy -2	1.75	Quaternary Basaltic deposits	2.03	4.45	3.04
17	Glenroy -3	2.00	Quaternary Basaltic deposits	6.04	2.14	2.87
18	Hastings	0.65	Tertiary Aged Sedimentary Deposits	1.59	3.80	2.55
19	Horsham-1	0.70	Quaternary Basaltic deposits	2.14	2.92	2.22
20	Horsham-2	1.70	Quaternary Basaltic deposits	0.40	4.42	2.57
21	Horsham-3	2.70	Quaternary Basaltic deposits	1.35	2.36	1.69
22	Horsham-4	3.75	Quaternary Basaltic deposits	0.55	3.89	2.32
23	Kilmore	1.00	Devonian Aged Sedimentary deposits	0.00	4.05	2.25
24	Kinglake	0.60	Devonian Sedimentary	0.20	4.93	2.80
25	Lyndhurst	1.00	Quaternary Alluvium	6.60	3.46	3.76
26	Lyndhurst	1.60	Quaternary Alluvium	2.57	4.53	3.23
27	Macleod	0.80	Silurian Sedimentary Clays	2.81	3.11	2.51
28	Melton West	0.50	Quaternary Basaltic deposits	4.03	7.98	5.55
29	Melton West	1.20	Quaternary Basaltic deposits	4.57	7.64	5.51
30	Mernda	0.40	Quaternary Basaltic Clays	0.93	4.39	2.70

Table 6 (cont'd) The summary of shrink-swelling test results

NO.	Location	Depth (m)	Soil types	ϵ_{sw} (%)	ϵ_{sh} (%)	I_{ss} (%/pF)
31	Ocean grove	0.75	Tertiary Aged Sedimentary deposits	0.22	4.31	2.46
32	Officer	0.65	Quaternary Aged Alluvium Deposits	0.00	3.44	1.91
33	Plumpton	0.70	Quaternary Aged Basaltic Residual Soils	0.62	3.66	2.21
34	Point Cook	0.60	Quaternary Basalt	8.01	7.08	6.16
35	Point Cook	0.65	Quaternary Aged Basaltic Residual Soils	9.89	5.13	5.60
36	Point cook	0.40	Quaternary Basaltic Clays	2.96	3.62	2.83
37	Point cook	0.60	Quaternary Basaltic Residual Soils	4.55	0.58	1.59
38	Port Campbell	1.70	Tertiary aged Port Campbell Limestone (Tmc)	1.93	13.00	7.76
39	South Morang	1.00	Quaternary Basaltic Clays	1.68	9.48	5.73
40	South Morang	0.75	Quaternary Aged Basaltic Residual Soils	11.40	5.38	6.16
41	Tarneit	0.40	Quaternary Basaltic Residual Soils	15.11	4.97	6.96
42	Tarneit	0.65	Quaternary Aged Basaltic Residual Soils	12.05	5.89	6.62
43	Templestowe	0.65	Silurian Aged Sedimentary deposits	1.39	1.59	1.27
44	Torquay	0.65	Torquay Group, sand/silt/clay deposits	0.00	5.50	3.06
45	Truganina	0.40	Quaternary Basaltic Clays	4.27	10.74	7.15
46	Tullamarine-1	0.20	Quaternary Basaltic deposits	0.82	1.63	1.13
47	Tullamarine-2	0.40	Quaternary Basaltic deposits	3.61	4.43	3.46
48	Tullamarine-3	0.75	Quaternary Basaltic deposits	3.02	3.10	2.56
49	Tullamarine-4	1.75	Quaternary Basaltic deposits	4.82	2.78	2.88
50	Vermont-1	0.75	Silurian Sedimentary	1.77	2.27	1.75
51	Vermont-2	1.45	Silurian Sedimentary	3.09	1.28	1.57
52	Warragul	0.70	Tertiary Basaltic	0.00	1.95	1.08
53	Warragul	0.75	Tertiary Aged Basaltic deposits	0.45	5.67	3.28
54	Williams Landing	0.65	Quaternary Aged Basaltic Residual Soils	19.37	9.76	10.80
55	Williamstown-1	0.30	Quaternary Basaltic deposits.	6.64	7.41	5.96
56	Williamstown-2	1.20	Quaternary Basaltic deposits.	8.20	7.28	6.32
57	Williamstown-3	2.00	Quaternary Basaltic deposits	2.22	11.53	7.02
58	Wollert	0.75	Quaternary Basaltic deposits	1.73	4.45	2.95
59	Wollert	0.75	Quaternary Aged Basaltic Residual Soils	7.66	8.56	6.88
60	Wollert	0.50	Quaternary Basaltic	10.02	6.46	6.37

4.2.2 Shrink-swell index map of Melbourne metropolitan area

Based on the results of laboratory tests, an interactive map, as shown in Figure 42, has been developed in this study. This map provides (1) the site location, (2) depths of soil samples collected, (3) site geology information, (4) shrink-swelling indices (I_{ss}), (5) liquid limit, (6) plastic index and (7) linear shrinkage. It covers a range of soil, geological and geographic conditions across the Melbourne metropolitan area. An interactive reactive soil map such as one present here is very useful for geotechnical engineers as it can be used to assess/calibrate the shrinkage index estimated on the visual-tactile basis by local engineers.

The shrink swell indices obtained from the laboratory tests have also been added to the geology map of Melbourne (scale: 1 – 250,000) as shown in Figure 43, which was downloaded from the Victorian Department of Primary Industries' GeoVic web site. It should be pointed out there are a number of samples marked as “out of range” in the map as they were collected from Geelong and Frankston areas..

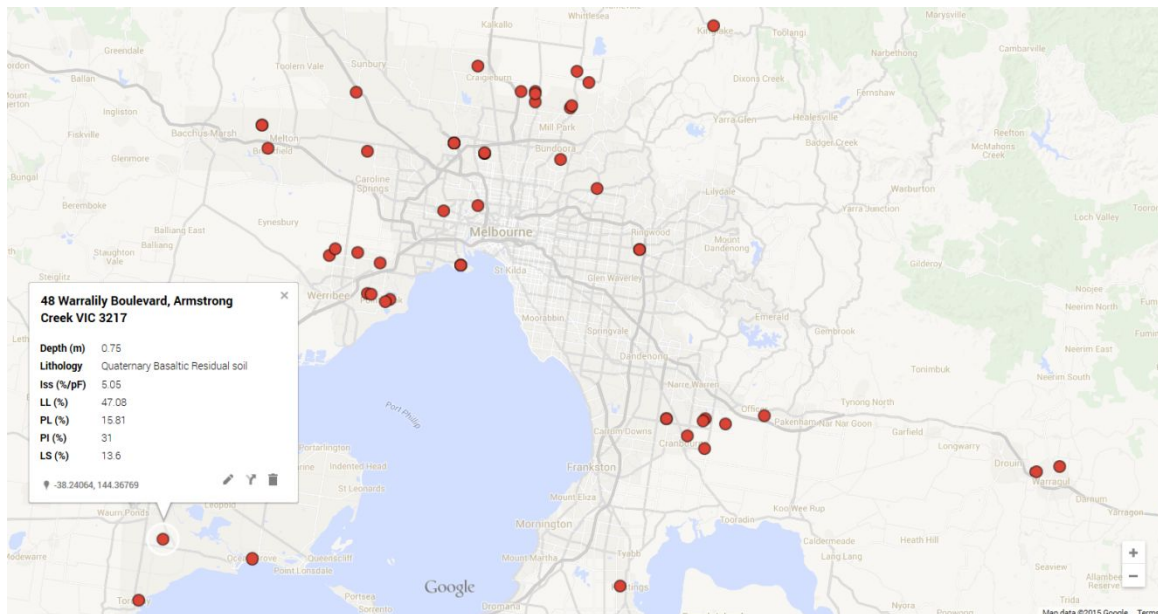


Figure.42 The interactive reactive soil map

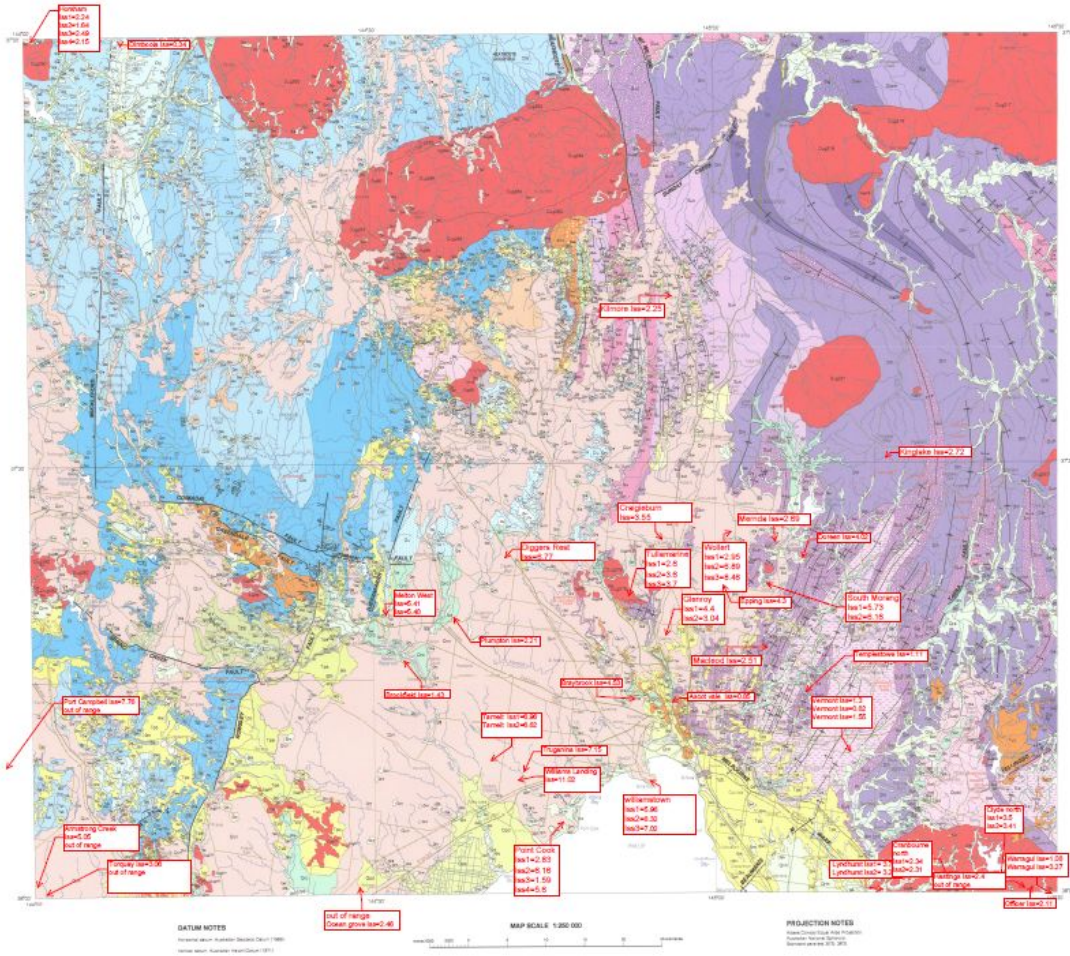


Figure.43 Geology map of Melbourne showing shrink-swell index

4.2.3 Statistics of soil reactivity

Shrink-swell test results obtained in Melbourne metropolitan area have been studied. Figure 44 illustrates the frequency (% of population) against ranges of shrink-swell index (%pF). The range has been set at 1.0 interval and from this point the frequency was calculated. It is apparent that the shrink-swell indices, Iss of clay soils in greater metropolitan area of Melbourne span a wide range from very low to very high values. The results reported in this study have confirmed that the variability of the reactivity of Melbourne soils may be difficult to assess by visual-tactile means. Therefore there is a need for frequent testing by the local consulting industry.

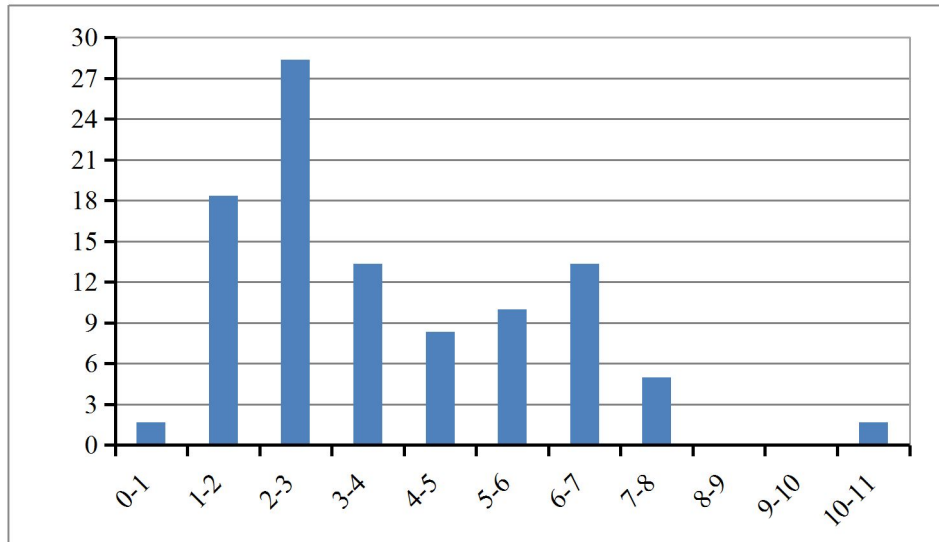


Figure.44 Histogram of shrink-swell indices

The shrink swell index may be directly used to evaluate the soil reactivity. Seddon (1992) suggested that Table 7 be used for site classification in Victoria. A histogram of site classification based Table 7 is presented in Figure 45. It is apparent that over 40% can be classified as M site.

Table.7 Soil classification based on the shrink-swell index

(Neilson et al. 1992)

AS 2870 Classification	I_{ss} (% strain / pF unit)
Class S	0.8-1.7
Class M	1.7-3.3
Class H	3.3-5.8
Class E	>5.8

Li *et al* (2014) also stated that a shrinkage index of 4%/pF would be regarded as a highly expansive soil, 6%/pF very highly expansive and 8%/pF, an extremely expansive soil. The results indicate that 20% of soil samples should be classified as very highly expansive ($I_{ss} \geq 6\%/pF$).

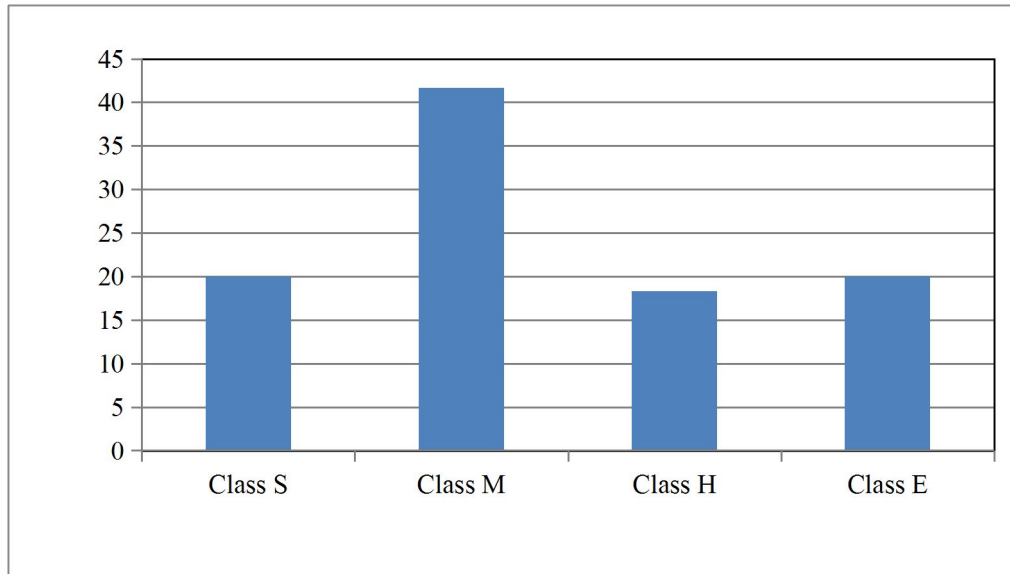


Figure.45 Sites classification based on the shrink-swell indices

Table 8 summarises the results of average shrink-swell index, I_{ss} and frequency by the different types of soil. The distribution of shrink-swell index for different soil types is given in Figure 46. It is not surprising that basalt and sedimentary soils dominate the soil data. Figure 47 illustrates the dominant nature of Quaternary basaltic soil in Melbourne area. The Quaternary basaltic soil comprises 45% of the samples collected/tested, and 67% when combined with Tertiary Basaltic and Quaternary Basaltic residual soil.

Table.8 The summarised results of average I_{ss} based on soil types

Soil types	Number of samples	Average of shrink swell index (%/pF)	Frequency (%)
Tertiary Aged Basaltic deposits	2	2.18	3.33
Tertiary Basaltic Residual Soils	1	1.87	1.67
Quaternary Basaltic residual soil	10	5.86	16.67
Quaternary Basaltic	27	3.97	45.00
Devonian Sediment	2	2.52	3.33
Silurian Sediment	5	2.22	8.33
Quaternary Alluvium	4	2.31	6.67
Tertiary Sediment	4	2.94	6.67
Quaternary Swamp and Lagoon	2	3.32	3.33
Torquay Group, sand/silt/clay deposits	1	3.06	1.67
Quaternary Colluvium	1	1.11	1.67
Tertiary aged Port Campbell Limestone (Tmc)	1	7.76	1.67

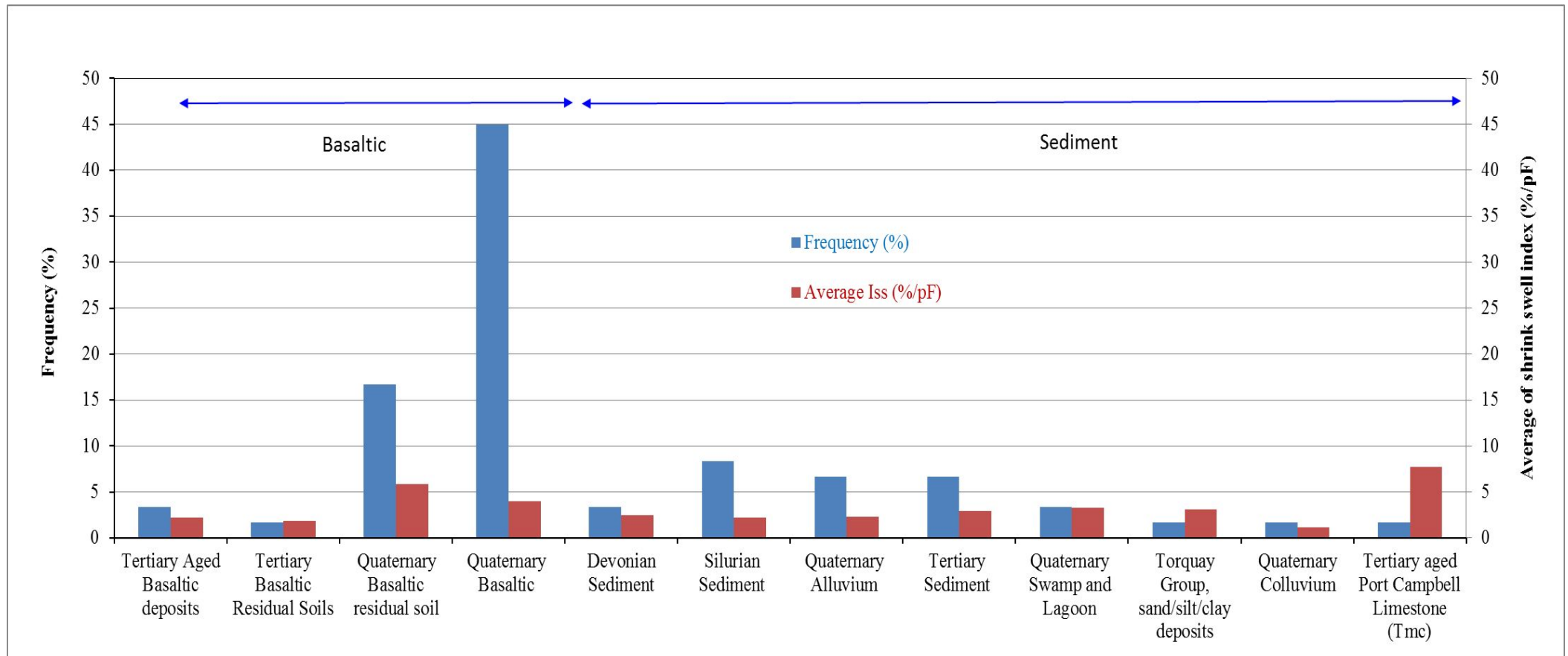


Figure.46 Distribution of shrink-swell indices for different soil types

Basaltic soil

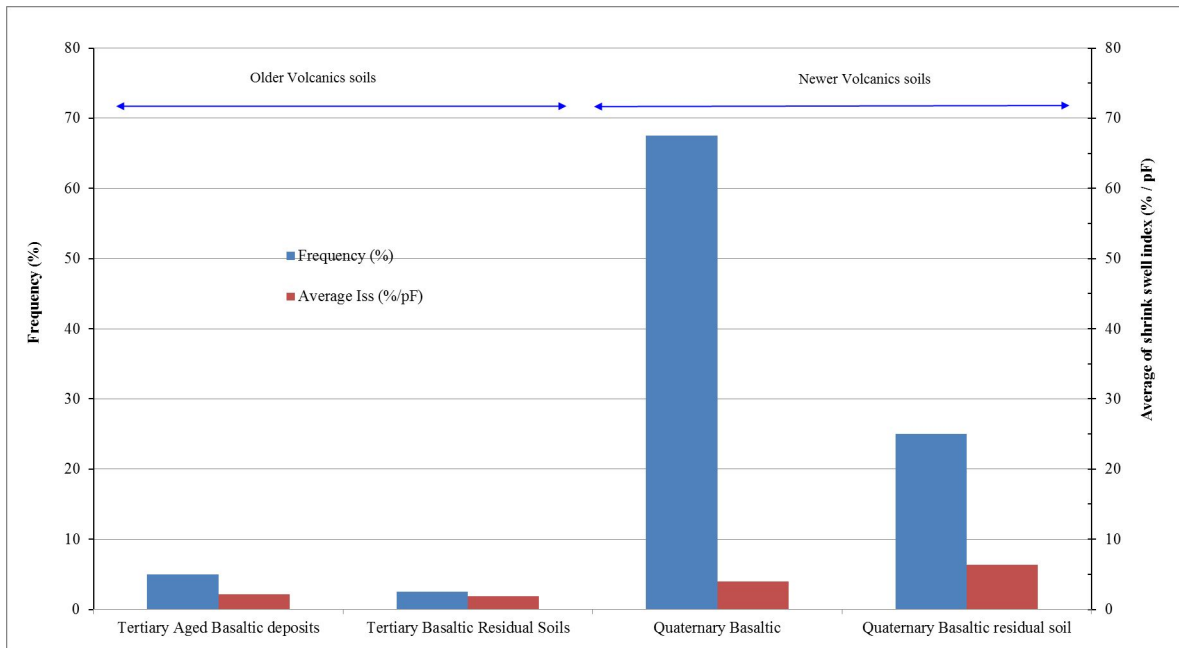


Figure.47 Distribution of shrink-swell indices for different basaltic soils

The basaltic clay soils can be divided into two groups based on the age of the soil, namely the Older Volcanic soil and the Newer Volcanic soil. The Older Volcanic includes tertiary basaltic while the New Volcanic comprises quaternary basaltic and quaternary residual basaltic. Figure 48 displays the average shrink-swell indices and frequency for different basaltic clays. It is evident that the New Volcanic clay soils are dominant in this study. The New Volcanic basaltic samples (mainly from the Western Suburb of Melbourne) are consistently recorded as having a shrink-swell index in the range of 4 to 6.5 %/pF, higher than that of the Old Volcanic basaltic. In Melbourne, the New Volcanic clay soils have caused the majority of footing problem for new residential buildings.

Sedimentary soil

Approximately 36% of the soil data can be grouped into another broad category, i.e. Sedimentary soils. Figure 49 shows the distribution of shrink-swell indices for different sedimentary soils. Compared basaltic soils, sedimentary soils have a lower range of I_{ss} and are generally classified as class M.

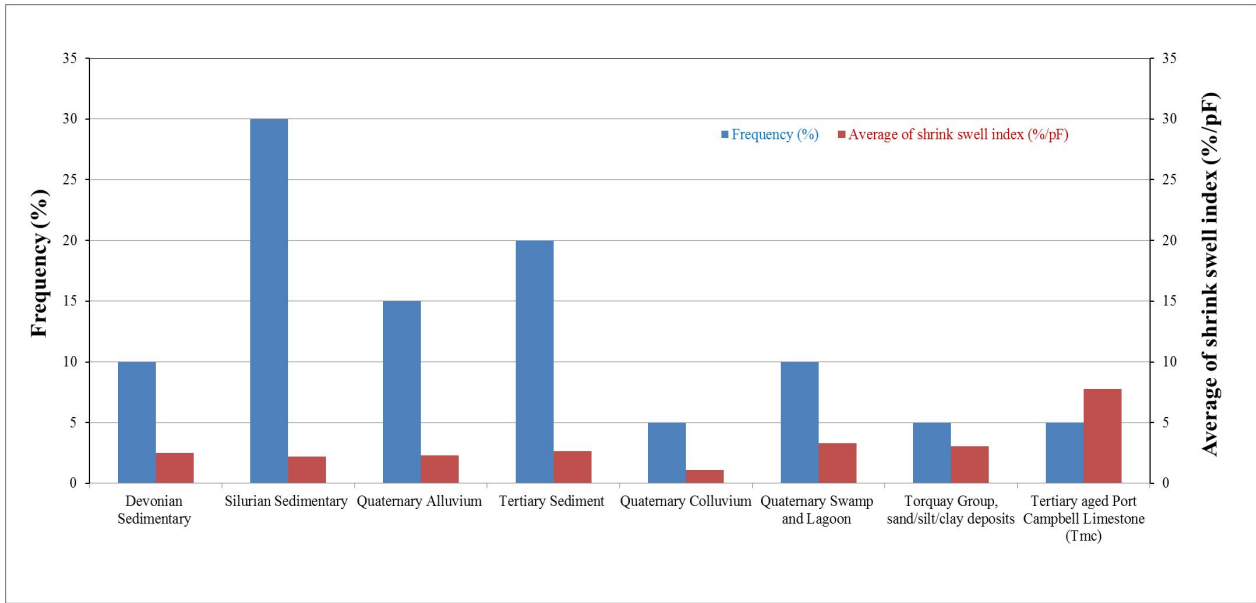


Figure.48 Distribution of shrink-swell indices for different sedimentary soils

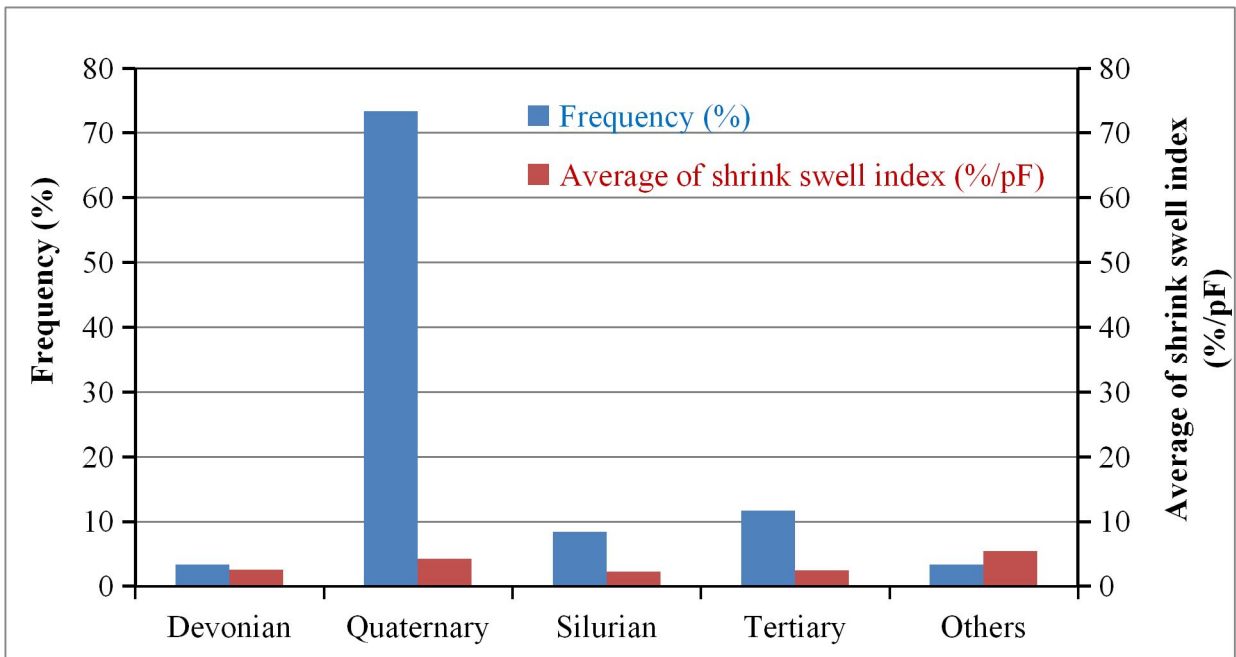


Figure.49 Distribution of shrink-swell indices for sedimentary soils based on the soil age

As shown in Figure 48, the Quaternary soils comprise 67% of the data with an average shrink-swell index of 4.3 %/pF, much higher than that of Silurian sedimentary soils.

4.3 Evaluation of the calculation method of shrink-swell index recommended by AS2870 (2011)

The following equation is recommended by AS2870 (2011) to calculate the shrink-swell index, I_{ss} :

$$I_{ss} = \frac{\varepsilon_{sh} + \frac{\varepsilon_{sw}}{\alpha_{eff}}}{\Delta u_{eff}} = \frac{\varepsilon_{sh} + \frac{\varepsilon_{sw}}{2.0}}{1.8} \quad \text{Equation. 20}$$

There is an inherent problem in the shrink-swell test. The swelling strain ε_{sw} is measured from a lateral restrained specimen in 1-dimension consolidation test in which the consolidation cell provides rigid lateral confinement to swelling soil while the shrinkage strain ε_{sh} is measured from an unrestrained core specimen which undergoes simultaneous shrink strain in 3-dimensions. Clearly ε_{sw} measured in the swelling test and ε_{sh} measured from the shrinkage test are incompatible and they could not simply be added together. Equation 22 has been empirically derived. A volume correlation factor of 2 is adopted for all soils to covert the axial strain measured in a sample swelling in one dimension to an equivalent axial strain in a sample swelling in three dimensions. Fityus (1996) tested two clays (an idealised soil and remoulded Maryland clay) and found that on average, the value of 2 gave reasonably uniform I_{ss} values over a range of soil moisture. The discussion of the appropriate correction volume factor are never stopped, Cameron (1989) found that the correction factor ranged between 1.7 to 2.15 base on the elastic theory and 66 core shrinkage tests. The second significant assumption of the shrink swell test is that the range of suction change corresponding to the soil volume change (from effective saturation to oven dry) is assumed to be equal to 1.8 pF units for all soils. This assumption was based on the collective experience of the AS2870 code committee (Fityus, 2005). The above two assumptions are used for all the types of soil and in all the conditions. However they have been questioned by some researchers and practitioners (Lopes, 2007).

In this study, an assessment of the appropriateness of using a suction range of 1.8 pF and a volume correlation factor of 2 has been made experimentally.

4.3.1 Evaluation of the correction factor for axial swelling test

In the shrink-swell test, vertical swelling strain is exaggerated by the imposition of complete lateral restraint using as a rigid steel ring. An empirical correction factor of 2 was introduced in AS2870 (2011) to reduce the axial swelling strain. Two testing methods can be used to experimentally assess the suitability of the assumed correction factor of 2: a three-dimensional volumetric shrinkage strain test method or a three-dimensional volumetric free swell test method. The volumetric swell test can be conducted in a triaxial cell. A soil specimen of 50 mm diameter and 100 mm height, wrapped in a rubber membrane, is placed between two porous stones and subject to moisture soaking from the bottom. The disadvantage of the volumetric swell test are (1) time-consuming, (2) the soil swelling is not completely free due to restraining effect of the rubber membrane.

In this study, three-dimensional volumetric shrinkage testing was conducted to assess the appropriateness of correction factor of 2. During the tests, a cylindrical soil specimen was subjected to air drying, and radial, axial and volumetric strains were regularly measured by using a digital imaging technology.

Two clays, namely Maryland clay and Braybrook clay were used in the laboratory tests. Maryland clay was selected because it was used in Fityus (1996) study.

The testing results of Maryland clay

Maryland clay is a heavy clay soil that occurs naturally at the Maryland expansive soil field site near Newcastle and can be described as a residual soil derived from a mudstone parent rock (Li *et al* 2007). The liquid limit, plastic index and linear shrinkage of Maryland clay are 65%, 41% and 18% respectively.

To avoid possible effects due to anisotropy and non-homogeneity within the soil samples, the same preparation procedure given in Li *et al* (2007) was adopted in this study. The soil was firstly dried in an oven at a temperature of about 100°C until it became non plastic and was then crushed. The crushed soil was wet sieved using a 425 μm screen to remove all coarse materials and vegetable matter. The soil passing through the sieve was combined and thoroughly mixed. The slurry was then placed into a special purpose pre-consolidation cell which used air pressure acting on a rubber membrane to apply a vertical pressure to the soil. After consolidating for 14 days under

vertical pressure, the soil sample was removed from the consolidation cell and trimmed into a cylindrical sample tube. A cylindrical, homogenised core specimen (50 mm diameter and 90 mm height) was rested on a smooth surface, allowing it to dry out for about 25 days. The volumetric, axial and radial strains were recorded four times per day during the first six days and then reduced to twice per day. The specimen was wrapped with plastic membrane overnight to alleviate moisture gradients within the specimen and reduce the risk of cracking.

In this study, the axial, radial and volumetric shrinkage of soil samples were measured by using a digit vernier calliper and checked with an Artec Spider hand-held 3D scanner (Figure 50). Together with the Artec Studio software, the Artec Spider 3D scanner can produces images of extremely high resolution (up to 0.1 mm) and superior accuracy (up to 0.05 mm), capturing up to 7.5 frames per second and processing 1 million points per second, which makes it faster than a conventional laser scanner (Artec Group, 2014). All the frames of a scan are automatically aligned, meaning that no complicated post-processing is required. Figure 51 shows 3D view of a cylindrical soil sample scanned with Spider 3D scanner.

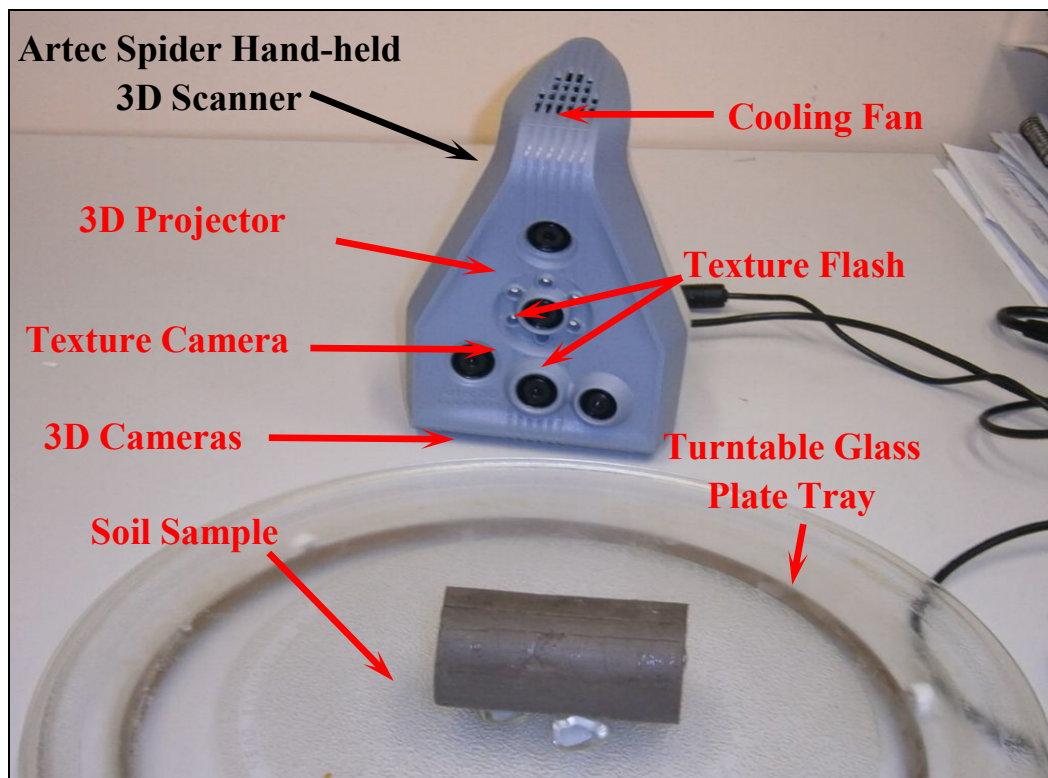


Figure.50 Artec Spider 3D scanner and soil sample used in the shrinkage testing

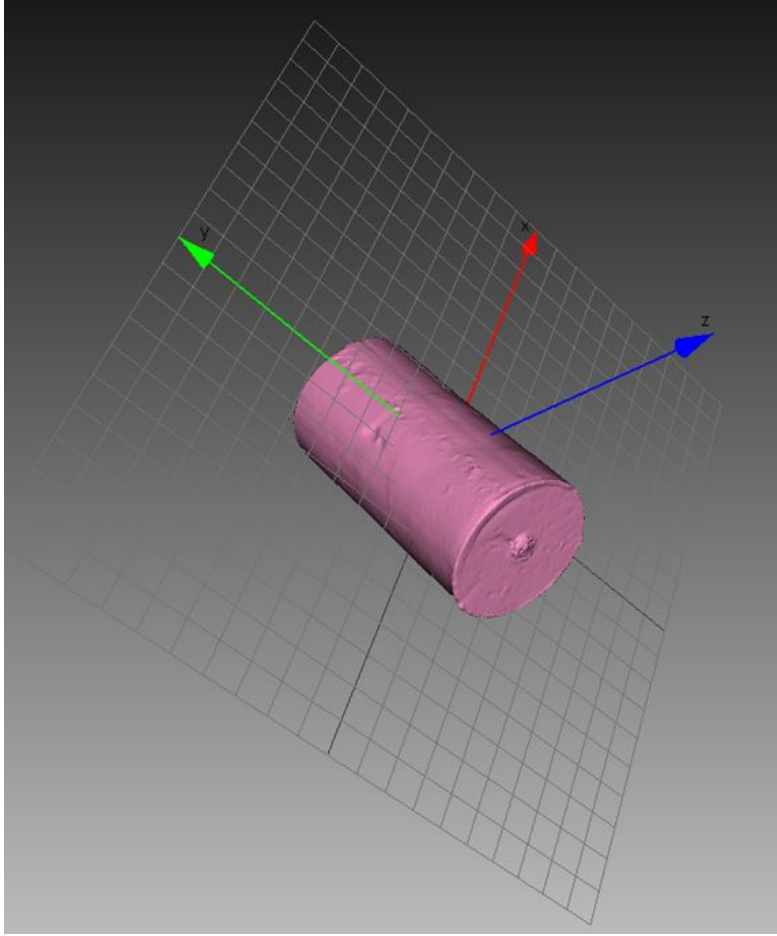


Figure.51 3D view of the cylindrical soil sample after scanned with Artec Spider 3D scanner

The changes in dimensions (length and diameter) and volume are plotted in [Figure 52](#). [Figure 53](#) depicts the vertical (axial), radial and volumetric shrinkage strains versus time elapsed during the shrinkage test. The majority of shrinkage movement and strains were observed within the first 80 hours, and subsequent shrinkage strains were continuously monitored until no shrinkage movement was observed. From [Figures 52-53](#), it can be seen that further drying of the Maryland clay beyond 5 pF did not result in any significant shrinkage strain, i.e. the strain at 5 pF may be taken as the Shrinkage Limit.

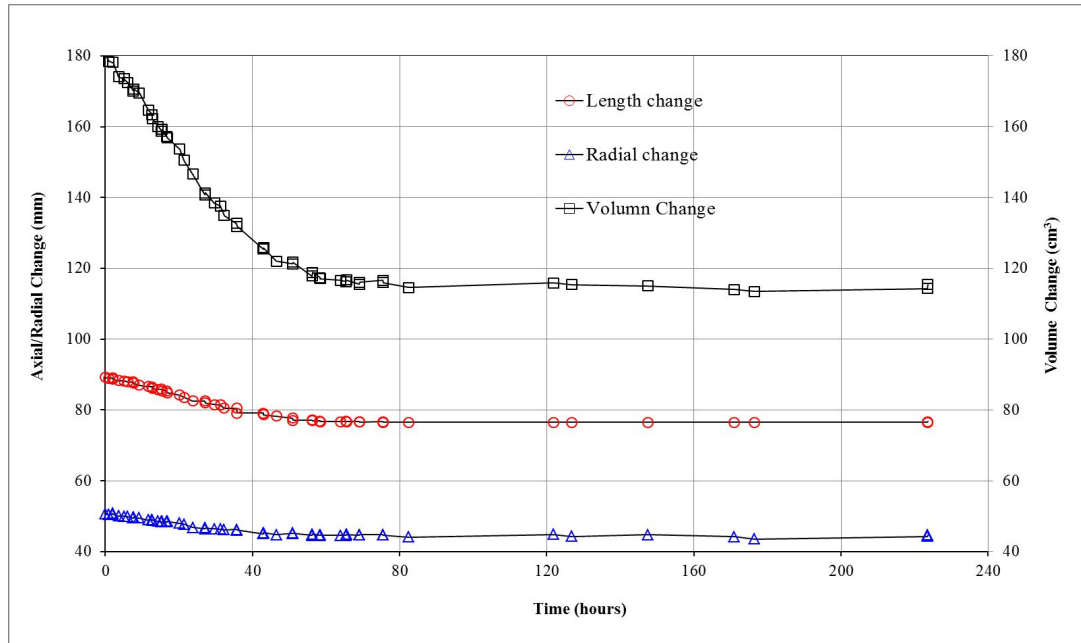


Figure.52 Shrinkage movement of the specimen (reconstituted Maryland clay) vs time

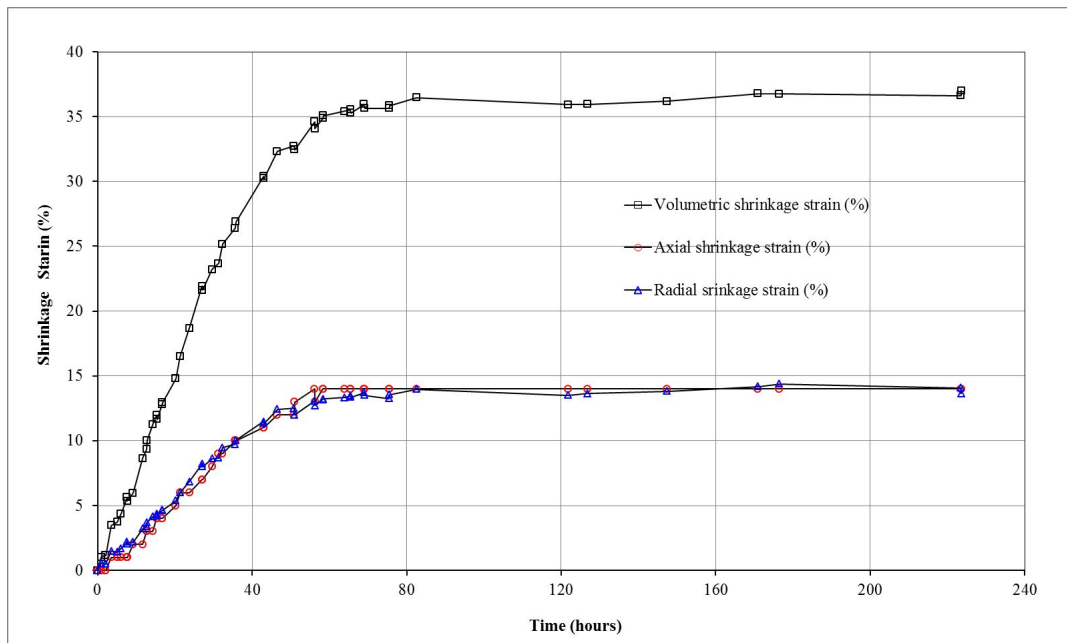


Figure.53 Volumetric, axial vs radial shrinkage strains vs time

The Soil-Water Characteristic Curve (SWCC) of Maryland clay, as shown in [Figure 54](#) was obtained using Fredlund SWCC device. The maximum matric suction allowed for the SWCC testes is limited to 1500 kPa (4.18 pF) due to the air entry value of the HAEV ceramic stone. In this study, the suction range above 1500 kPa (i.e., total suction) was measured by using a Decagon WP4 Dewpoint Potentiometer which uses the chilled-mirror dewpoint technique to measure total suction.

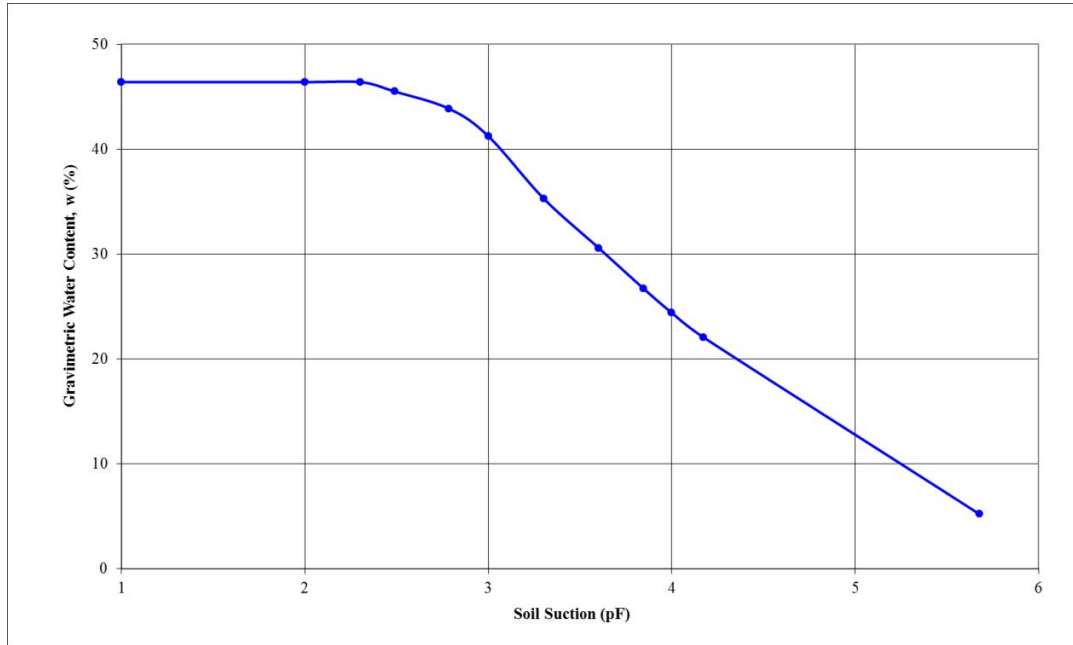


Figure.54 Soil-Water Characteristic Curve (SWCC) of Maryland clay

In [Figure 55](#), change in soil suction and water content is presented versus elapsed time. It can be seen that during the first 36 hours, the curve of suction *vs* time is almost a straight line before reaching 4.13 pF. After 80 hours, the curve rose continuously but became flatter until it reached a maximum value of 5.65 pF (air drying limit). The plots of the shrinkage movement of Maryland clay specimen versus soil suction and water content are presented in [Figures 56 to 57](#) respectively. From [Figure 58](#), it can be seen that the strains *vs* suction relationships are near-linear from 2.33 pF (initial total suction at near saturation) to 4.5 pF. From 4.5 pF to 5.65 pF (air dry), the slope of strains *vs* suction curves become much smaller. Volumetric and axial strains in the range of 4.5 – 5.65 pF are still substantial, which are 12.6% and 14.3% of the total volumetric and axial strains.

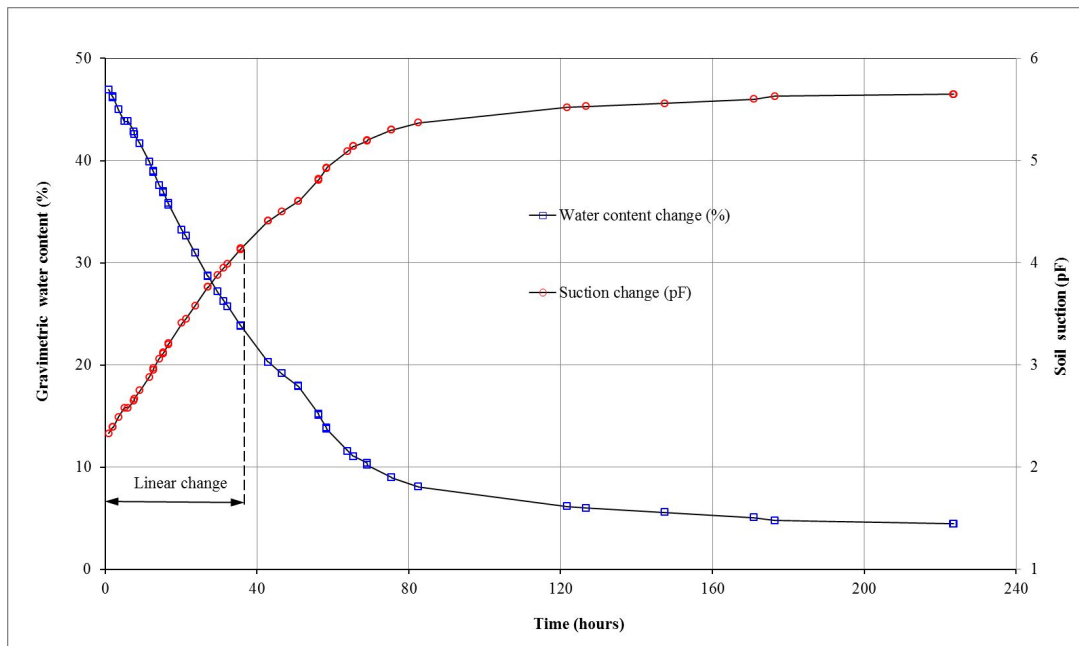


Figure.55 Soil suction and gravimetric water content vs time

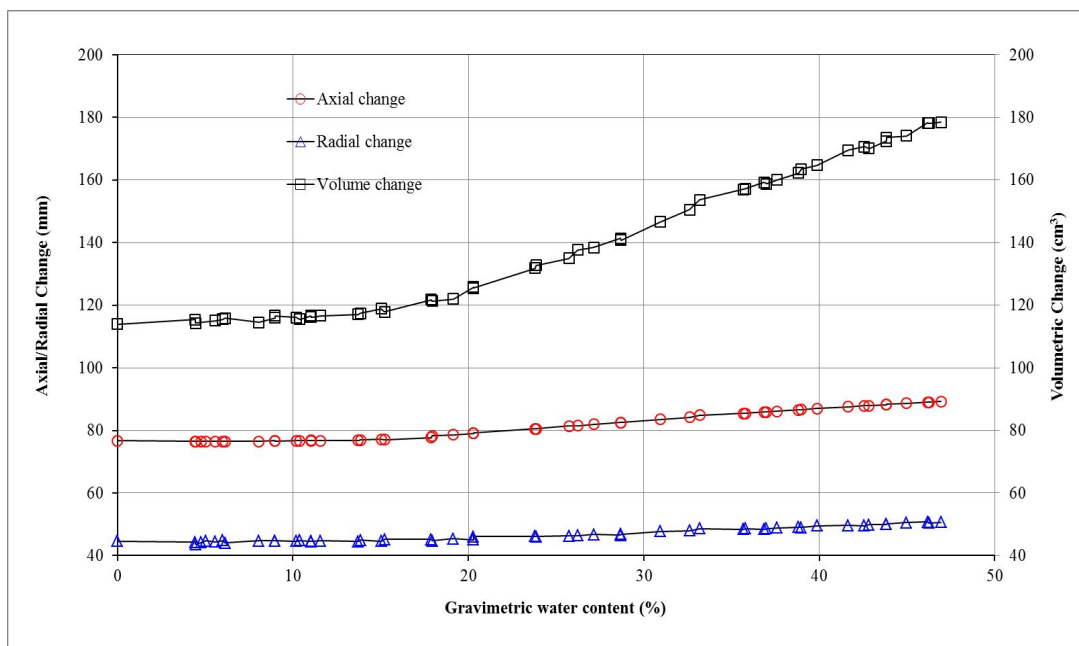


Figure.56 Shrinkage movement of the specimen vs soil water content change

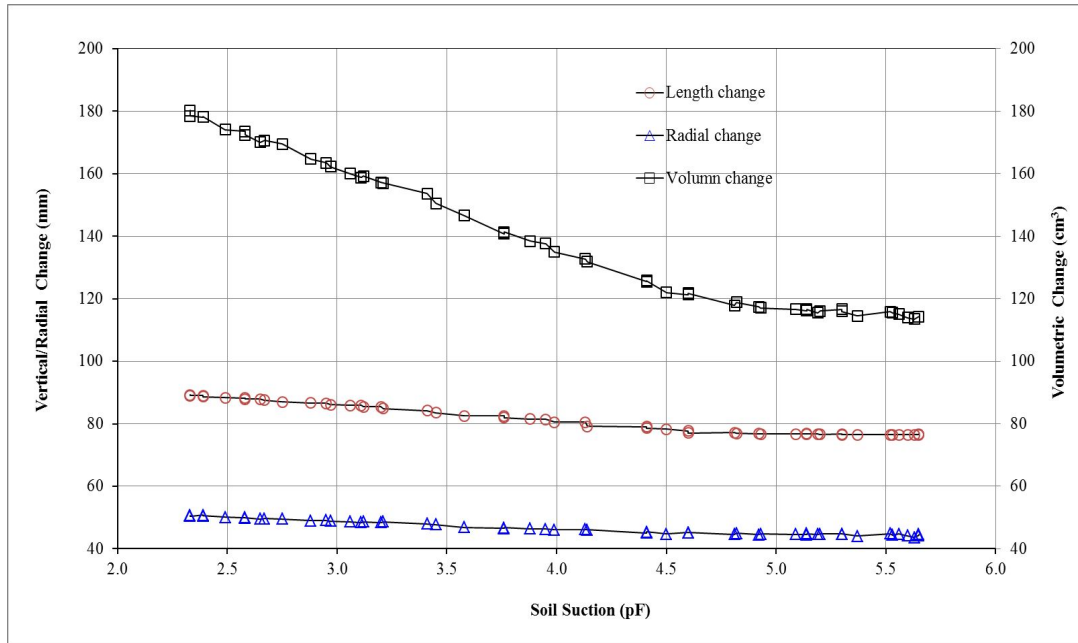


Figure.57 Shrinkage movement of the specimen vs soil suction change

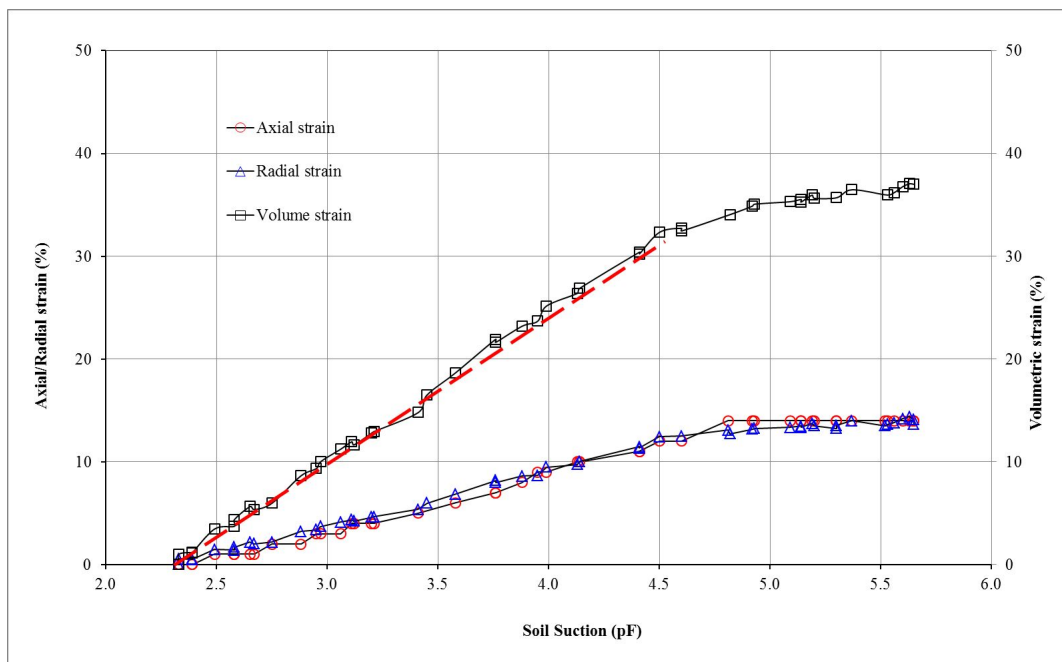


Figure.58 Volumetric, axial and radial shrinkage strains vs soil suction change

The plots of volumetric strain versus axial strain, volumetric strain versus radial strain and axial strain versus radial strain are shown in Figures 59 – 61. From Figure 59, it can be seen that although the measured data is slightly scattered, the correlation of volumetric strain with axial strain is very well, yielding an average slope value close to 2.62. This value is 30% higher than the lateral restraint factor of 2 assumed in AS2870 (2011). It is interesting to note that the ratio of

volumetric strain to radial strain is also equal to 2.65 (Figure 60), i.e. the relation of axial and radial strain can be expressed as a straight line with a slope of approximately 1, as shown in Figure 61).

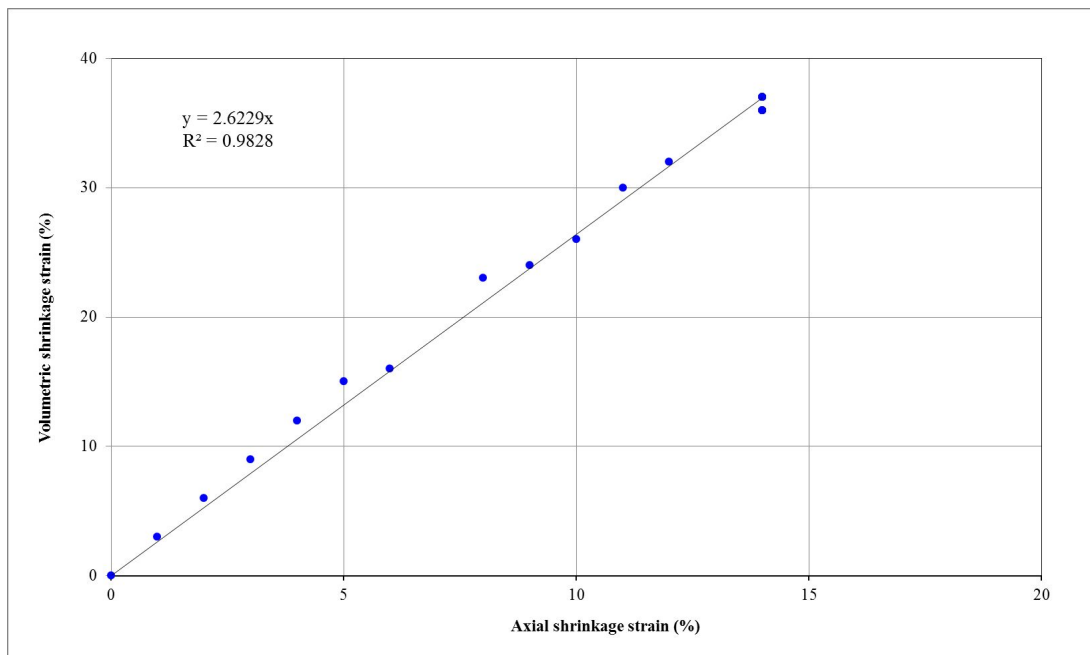


Figure.59 Volumetric shrinkage strain vs axial (or vertical) shrinkage strain (reconstituted Maryland clay)

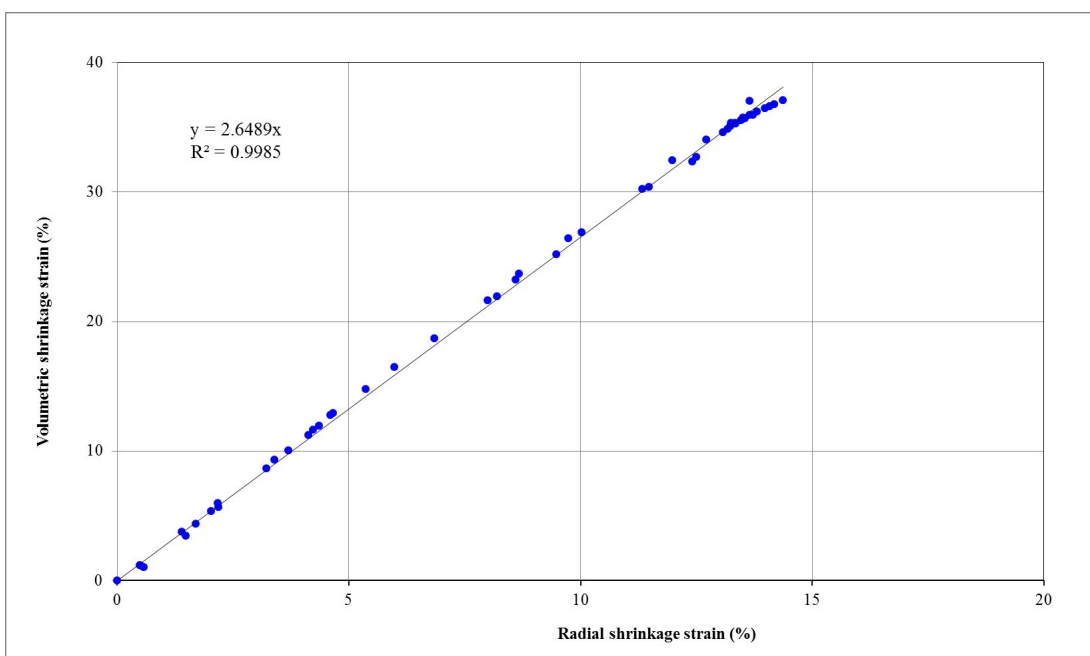


Figure.60 Volumetric shrinkage strain vs radial shrinkage strain (reconstituted Maryland clay)

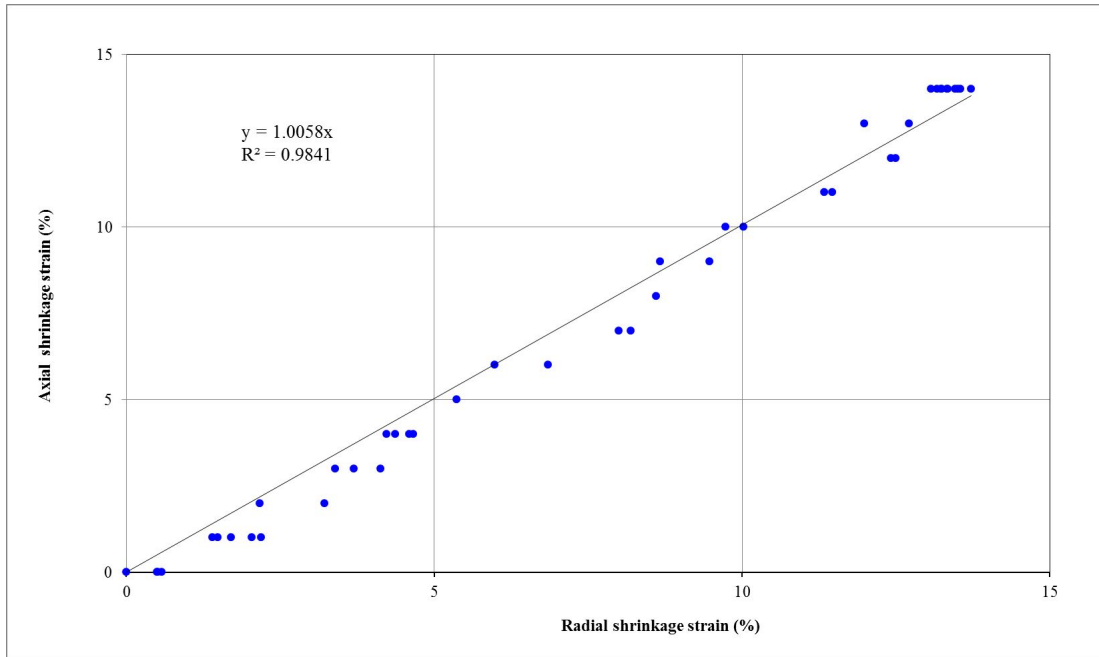


Figure.61 Axial shrinkage strain vs radial shrinkage strain (reconstituted Maryland clay)

The testing results of Braybrook clay

The shrinkage tests were also conducted on the remoulded Braybrook clay. Soil samples were obtained from an expansive soil field site in Braybrook, a Western suburb of Melbourne. The clay content of soil sample used in laboratory testing is approximately 36%. The liquid limit, plastic index and linear shrinkage of Braybrook clay are 80.75%, 18.02% and 22.10% respectively. The soil was first crushed and sieved using a 425 μm screen to remove all coarse sands and vegetable matter. The soil, prepared to two different initial moisture contents (12% and 24% respectively), was compacted in a 50 mm diameter tube in 5 equal layers, about 20mm in each layer. The sealed tubes were stored horizontally in a constant temperature room for two weeks to allow moisture throughout the sample to equilibrate. Three-dimensional volumetric shrinkage testing was then performed. Results for the tests performed on remoulded Braybrook clay samples are shown in [Figures 62-73](#).

The following observations can be made from [Figures 62 -73](#):

- 1) The majority of shrinkage movement and strain occurred with the first 120 hours for the remoulded Braybrook clay samples;

- 2) There was a near linear relationship between shrinkage movement and change in gravimetric water content;
- 3) When the initial water content of the specimen decreased from 24% to 12%, the total volumetric, axial and radial shrinkage strains were reduced 55%, 52% and 54% respectively;
- 4) For the remoulded Braybrook clay specimen with an initial water content of 24%, the ratio of volumetric strain to axial strain is 2.86, 43% higher than the factor of 2 suggested by AS2870 (2011);
- 5) For the specimen with an initial water content of 12%, the ratio of volumetric strain to axial strain is 2.62, same as that obtained from the reconstituted Maryland clay;
- 6) The relationship between axial and radial strains can be approximated by a straight line passing through the origin with a slope of 1.04 ($w_{ini} = 24\%$) and 1.12 ($w_{ini} = 12\%$), respectively, indicating that they are nearly equal to each other.

The results reported here suggest that the values of correction factor between 2.6 and 2.9 may be more appropriate and less conservative. It is suggested that more tests on different soils be conducted to assess the appropriateness of the assumed factor of 2.

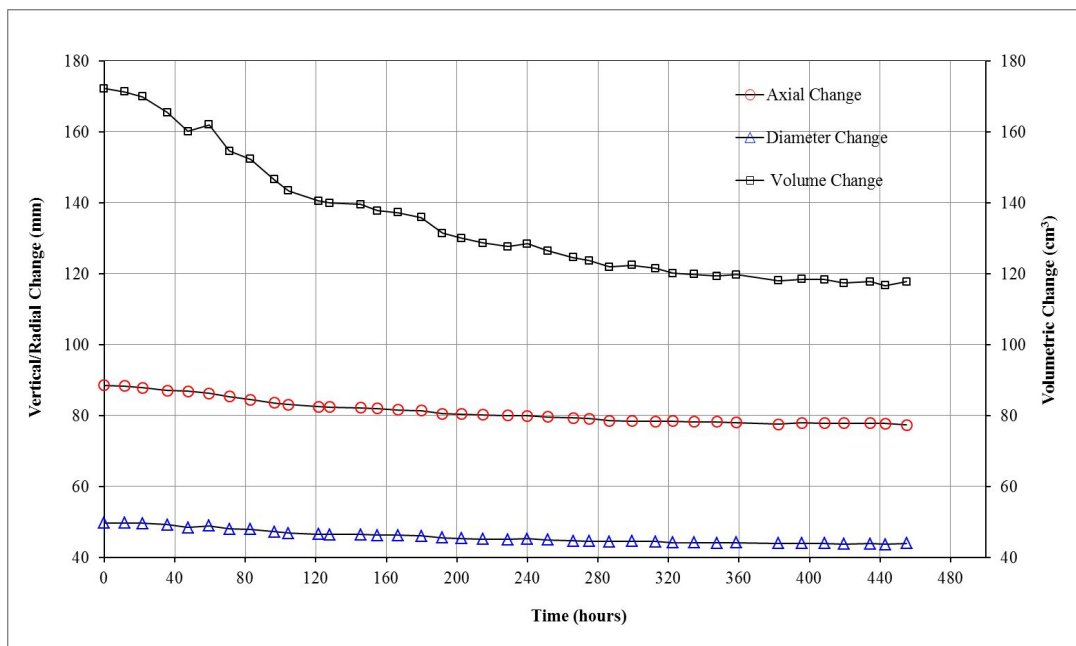


Figure.62 Shrinkage movement of the specimen (remoulded Braybrook clay, $w_{ini} = 24\%$) vs time.

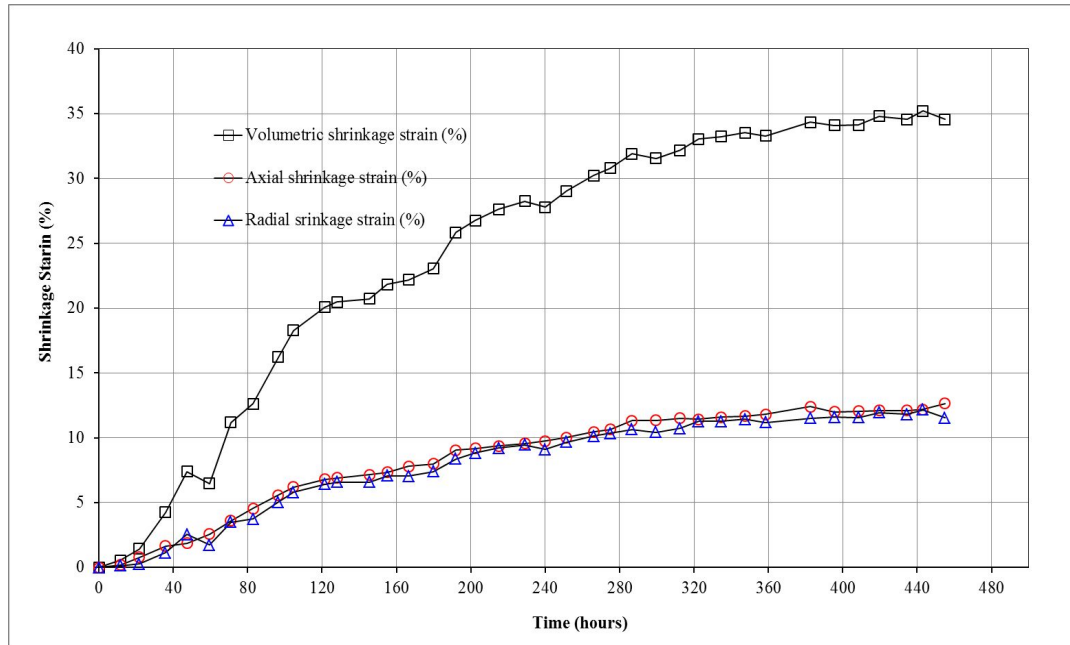


Figure.63 Volumetric, axial and radial shrinkage strains of the specimen (remoulded Braybrook clay, $w_{ini} = 24\%$) vs time.

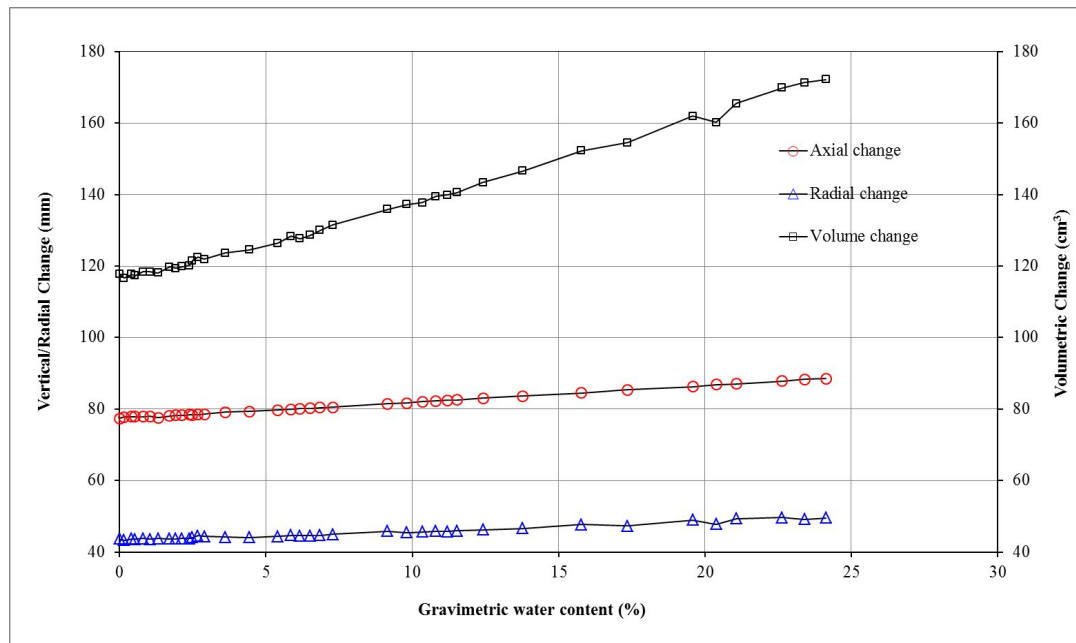


Figure.64 Shrinkage movement of the specimen (remoulded Braybrook clay, $w_{ini} = 24\%$) vs soil water content change

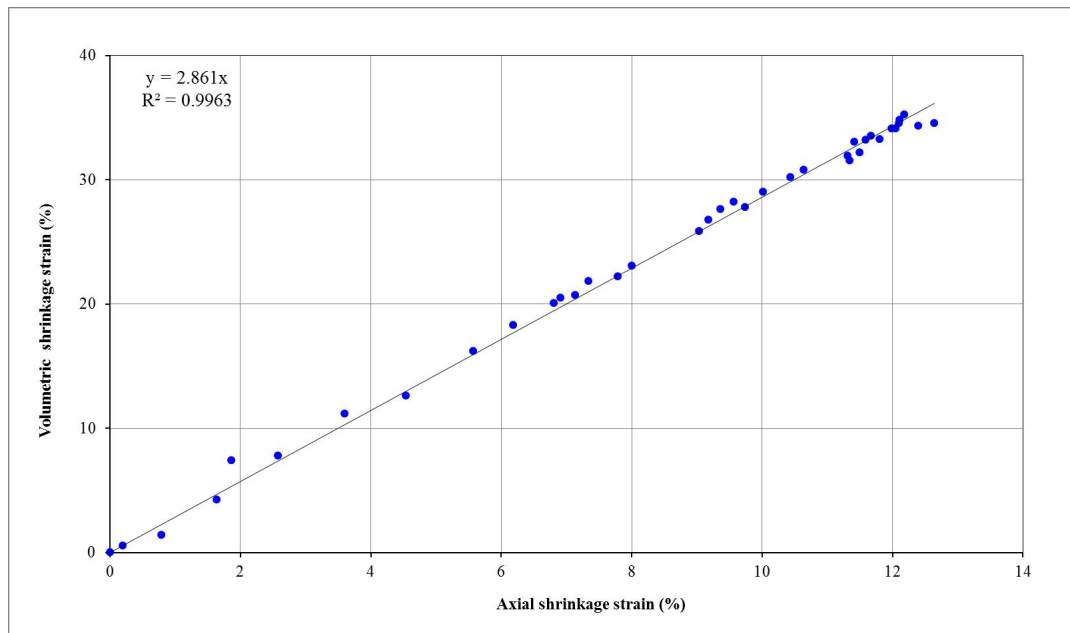


Figure.65 Volumetric shrinkage strain vs axial shrinkage strain (remoulded Braybrook clay, $w_{ini} = 24\%$)

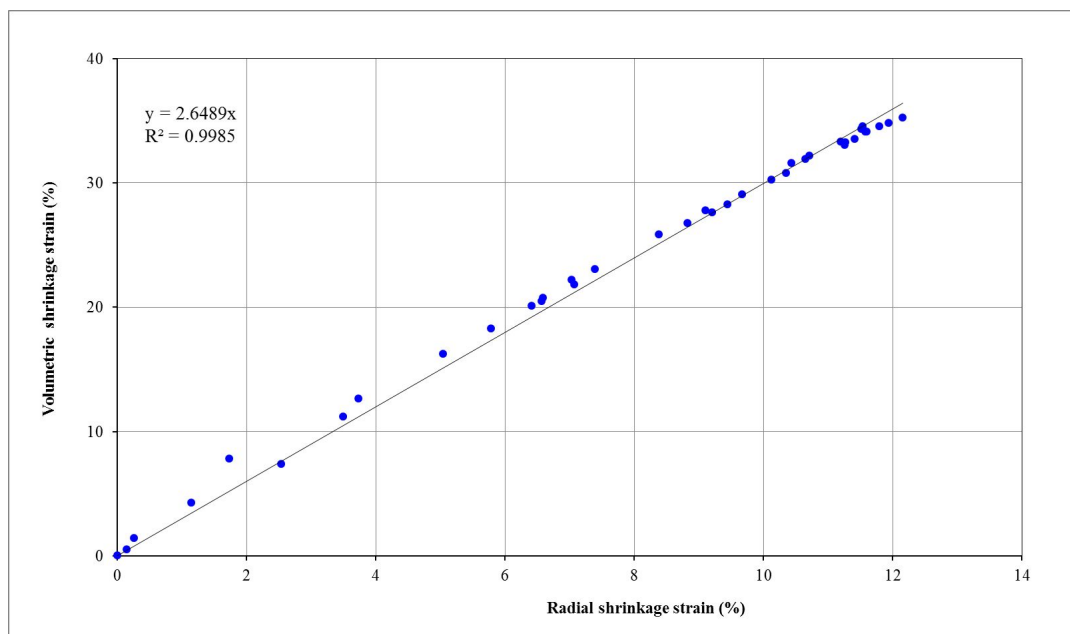


Figure.66 Volumetric shrinkage strain vs radial shrinkage strain (remoulded Braybrook clay, $w_{ini} = 24\%$)

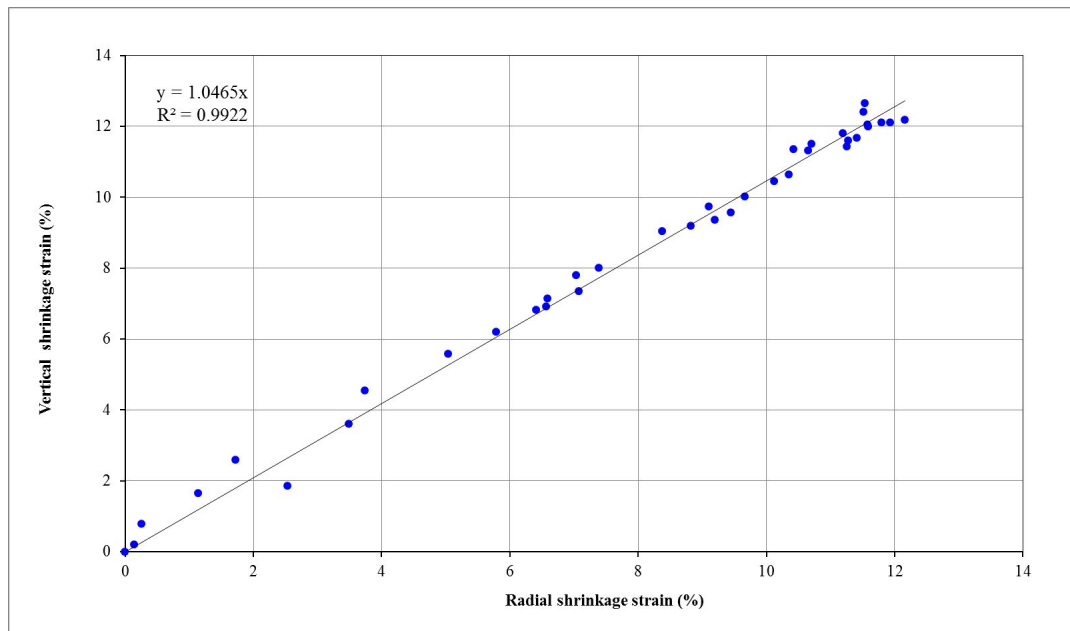


Figure.67 Axial shrinkage strain vs radial shrinkage strain (remoulded Braybrook clay, $w_{ini} = 24\%$)

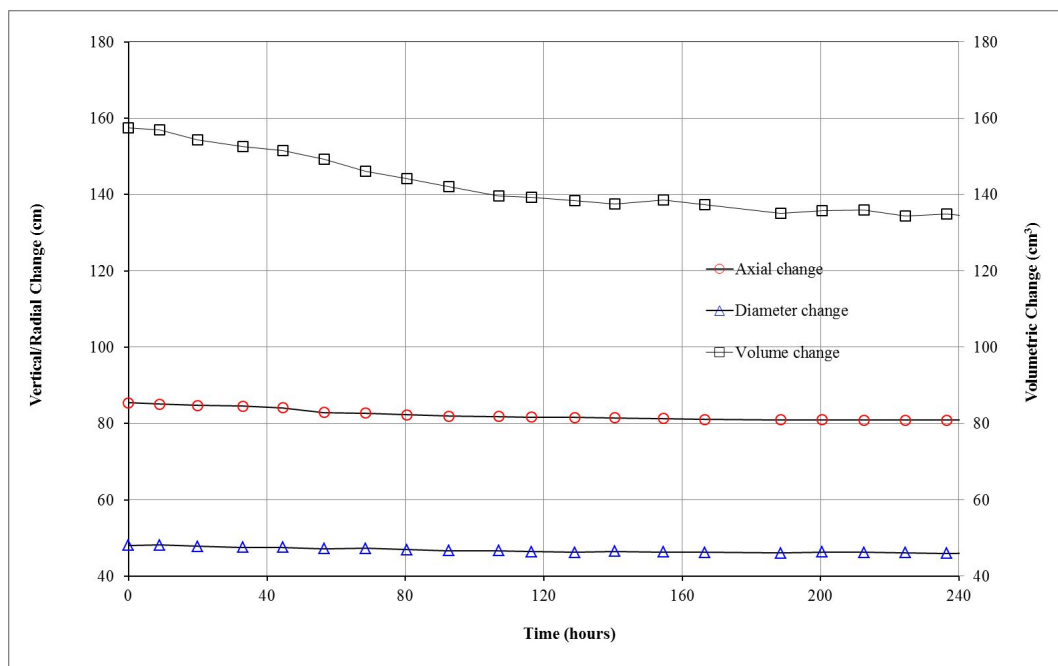


Figure.68 Shrinkage movement of the specimen (remoulded Braybrook clay, $w_{ini} = 12\%$) vs time.

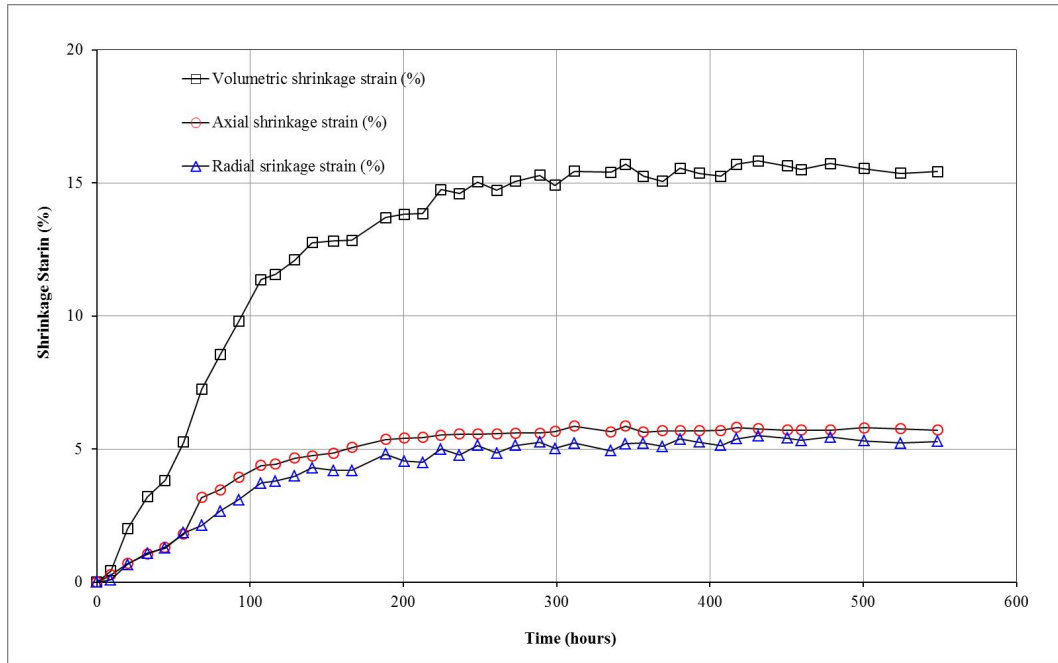


Figure.69 Volumetric, axial and radial shrinkage strains of the specimen (remoulded Braybrook clay, $w_{ini} = 12\%$) vs time.

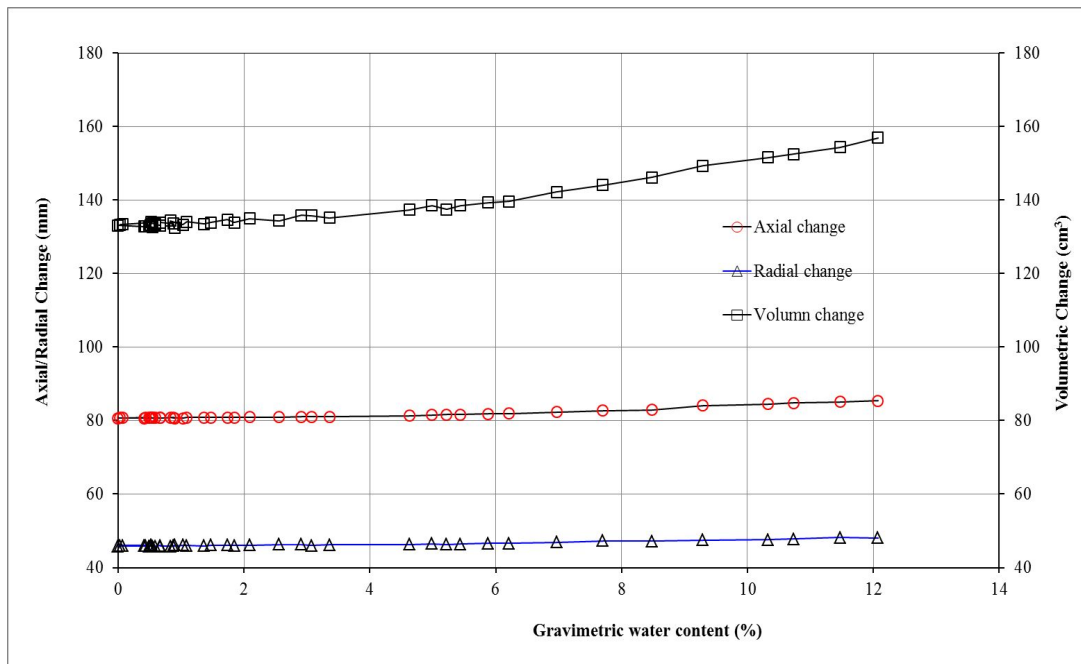


Figure.70 Shrinkage movement of the specimen (remoulded Braybrook clay, $w_{ini} = 12\%$) vs soil water content change

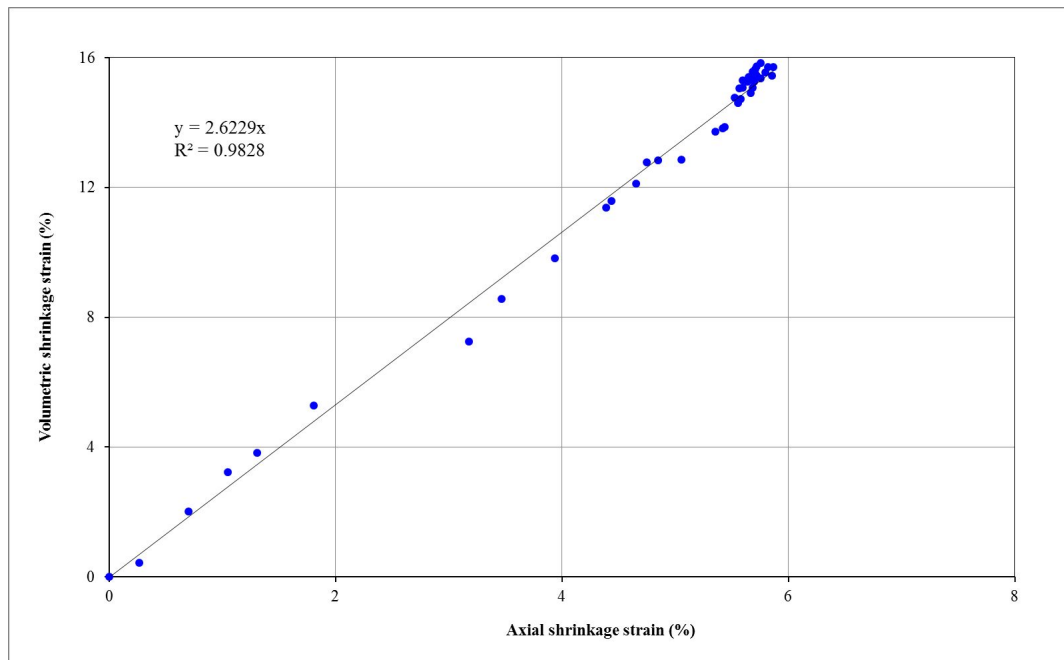


Figure.71 Volumetric shrinkage strain vs axial shrinkage strain (remoulded Braybrook clay, $w_{ini} = 12\%$)

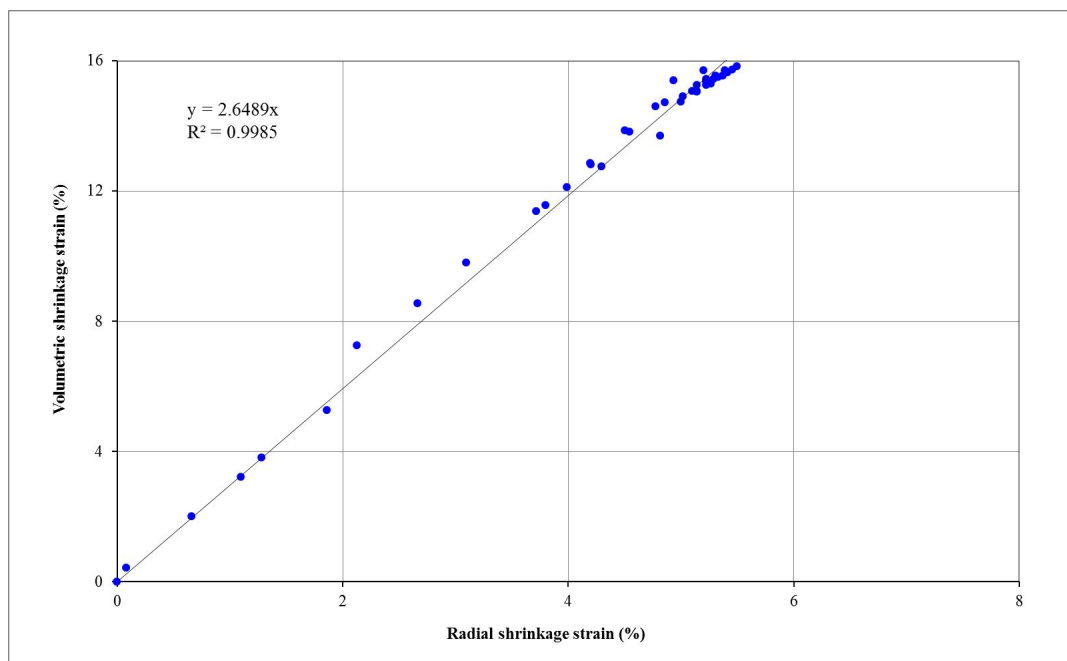


Figure.72 Volumetric shrinkage strain vs radial shrinkage strain (remoulded Braybrook clay, $w_{ini} = 12\%$)

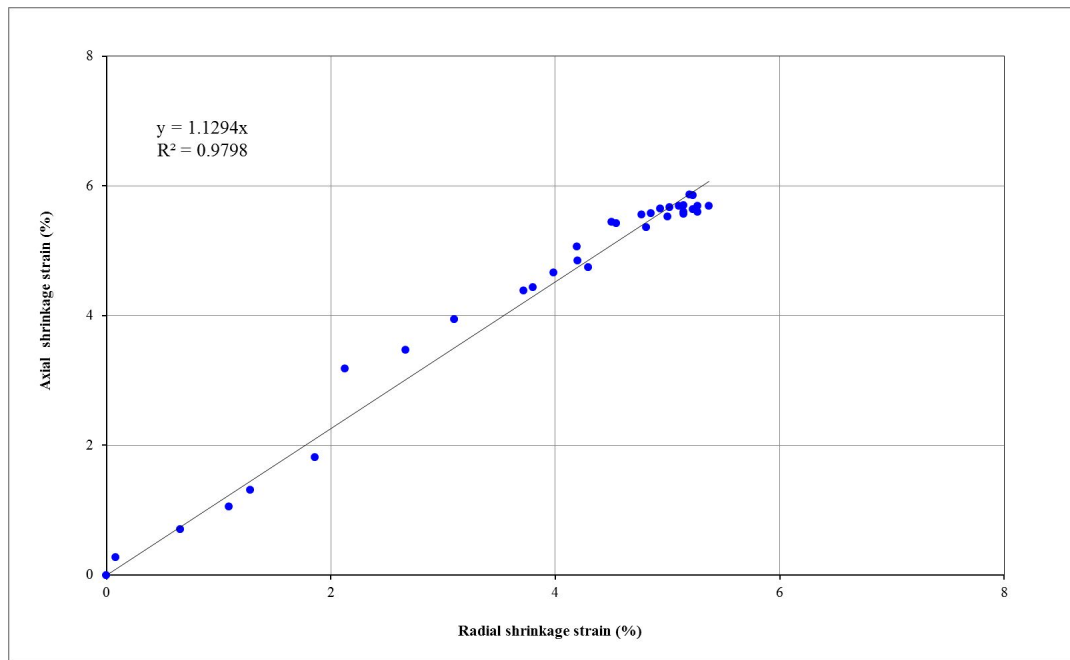


Figure.73 Axial shrinkage strain vs radial shrinkage strain (remoulded Braybrook clay, $w_{ini} = 12\%$)

4.3.2 The factor of 1.8 pF

Another significant assumption introduced in AS1289.7.1.1 (2003) is that magnitude of suction change over which the volume change takes place is assumed to be equal to 1.8 pF units for all soils. This assumption effectively bypasses the need to measure suction of soil specimen during the shrink-swell testing. However the 1.8 pF range is assumed rather than measured and AS1289.7.1.1 (2003) does not explain why this value is adopted in the calculation. There is some debate as to whether this value is appropriate for all Australian soils (Lopes, 2007; Hargreaves, 2008). Cameron (1989) pointed out that the denominator of 1.8 represented the likely suction range over which volume change were almost linearly proportional to the suction change. Base on Table 9, Fityus et al (2005) suggested that it was reasonable to assume suction to vary between 2.2-2.5 pF to 4.0-4.4 pF, i.e. assuming that the volume change takes place between the “wilting point” to a water content close to saturation.

Taking into account the osmotic suction component, Fityus and Cameron (2007) states “Hence it is considered that during the shrink swell tests all of the recorded volume change takes place between total suction of 3.7 and 5.5 pF: an interval of 1.8 pF units.”

Table.9 Benchmark intercept suction values

Benchmark (pF)	Soil state	Reference
4.2	Wilt point	Wray, 1998
4.0 to 4.4	Wilt point	Cameron, 2001
6.5 to 7.0	Zero water content	McKeen, 1992
6.7	Zero water content	Mitchell and Avalle, 1984
2.2 to 2.5	Field saturation	Fredlund & Rahardjo, 1993

Lopes (2006) contends that the shrink-swell test is inherently flawed because of the suction range assumed to be covered by the shrink-swell test and the linearity of volumetric strain over this particular suction change. However, the evidence provided in Lopes (2006) is based on the quoted soil suction sign posts (Table 10), not based on any actual suction measurement.

Table.10 Soil suction “sign posts”

(After Lopes, 2006)

Suction (pF)	Soil State	References
6.5 to 7.0+	Oven Dry	Lytton, Leeper, Uren, Mitchell et al
6.0	Air Dry	Lytton, Leeper, Uren
5.5	Shrinkage Limit	McKeen, Mitchell et al
3.2 to 3.5	Plastic Limit	Lytton
3.0	0.4 Liquid Limit	Driscoll
1.5 to 2.0	Swell Limit	McKeen

In this study, the total suction of twenty four soil samples was measured not only before the shrink-swell tests (i.e. initial suction) but also after swelling test and air dry. Table 11 summarizes the values of soil suction measured before and after shrink-swell tests. The range of soil suction change is plotted in Figure 74. A statistical summary of the soil suction changes is presented in Table 11.

Table.11 Summery of soil suction change in the shrink- swell tests

NO.	Soil samples	Lithology	Initial water content (%)	Initial suction (pF)	Water content after Swelling test (%)	Soil suction after swelling test (pF)	Soil suction after shrinkage test air dry (pF)	The soil suction change (pF)
1	Braybrook	Quaternary Basaltic	33.67	4.45	35.42	3.40	5.81	2.41
2	Armstrong Creek	Quaternary Basaltic Residual	20.43	3.86	22.00	3.72	5.72	2.00
3	Tarneit	Quaternary Basaltic Residual	27.58	3.65	41.52	3.07	5.99	2.92
4	Point Cook	Quaternary Basaltic Residual	24.42	4.89	40.53	3.42	5.98	2.56
5	South Morang	Quaternary Basaltic Residual	23.01	4.39	34.78	3.58	5.40	1.82
6	Wollert	Quaternary Basaltic Residual	25.63	4.77	40.83	3.27	6.05	2.78
7	Williams Landing	Quaternary Basaltic Residual	25.08	4.38	46.16	3.47	6.11	2.64
8	Plumpton	Quaternary Basaltic Residual	17.60	3.50	29.46	3.02	5.77	2.75
9	Diggers rest	Quaternary Basaltic Residual	34.73	3.78	40.05	3.59	6.08	2.49
10	Tarneit	Quaternary Basaltic Residual	26.50	3.81	35.08	3.71	6.09	2.38
11	Point Cook	Quaternary Basaltic Residual	27.76	3.79	42.79	3.58	6.32	2.74
12	Warragul	Tertiary Basaltic	31.68	3.70	31.97	3.16	6.02	2.86
13	Clyde north	Tertiary Basaltic	28.14	4.05	30.41	3.43	6.12	2.69
14	Cranbourne north	Quaternary Swamp and Lagoon	27.80	3.64	29.13	3.38	6.15	2.77
15	Clyde north	Quaternary Swamp and Lagoon	27.67	3.50	30.22	3.37	6.10	2.73
16	Officer	Quaternary Alluvium	14.86	3.28	15.89	2.97	5.54	2.57
17	Dimboola,	Quaternary Alluvium	13.44	4.11	21.83	2.82	6.04	3.22
18	Ocean grove	Tertiary Sedimentary	31.54	3.71	36.67	3.14	6.01	2.87
19	Hastings	Tertiary Sedimentary	27.73	4.56	31.37	3.83	5.78	1.95
20	Cranbourne North	Tertiary Sedimentary	19.90	3.97	24.08	2.96	6.01	3.05
21	Kilmore	Devonian Sedimentary	17.07	3.77	23.92	3.38	5.75	2.38
22	Templestowe	Silurian Sedimentary	14.42	4.35	18.63	3.51	5.91	2.41
23	Torquay	Torquay Group, sand/silt/clay	34.92	3.57	41.56	2.94	4.88	1.94
24	Port Campbell	Tertiary aged Port Campbell Limestone	44.18	3.54	44.50	3.04	5.87	2.83

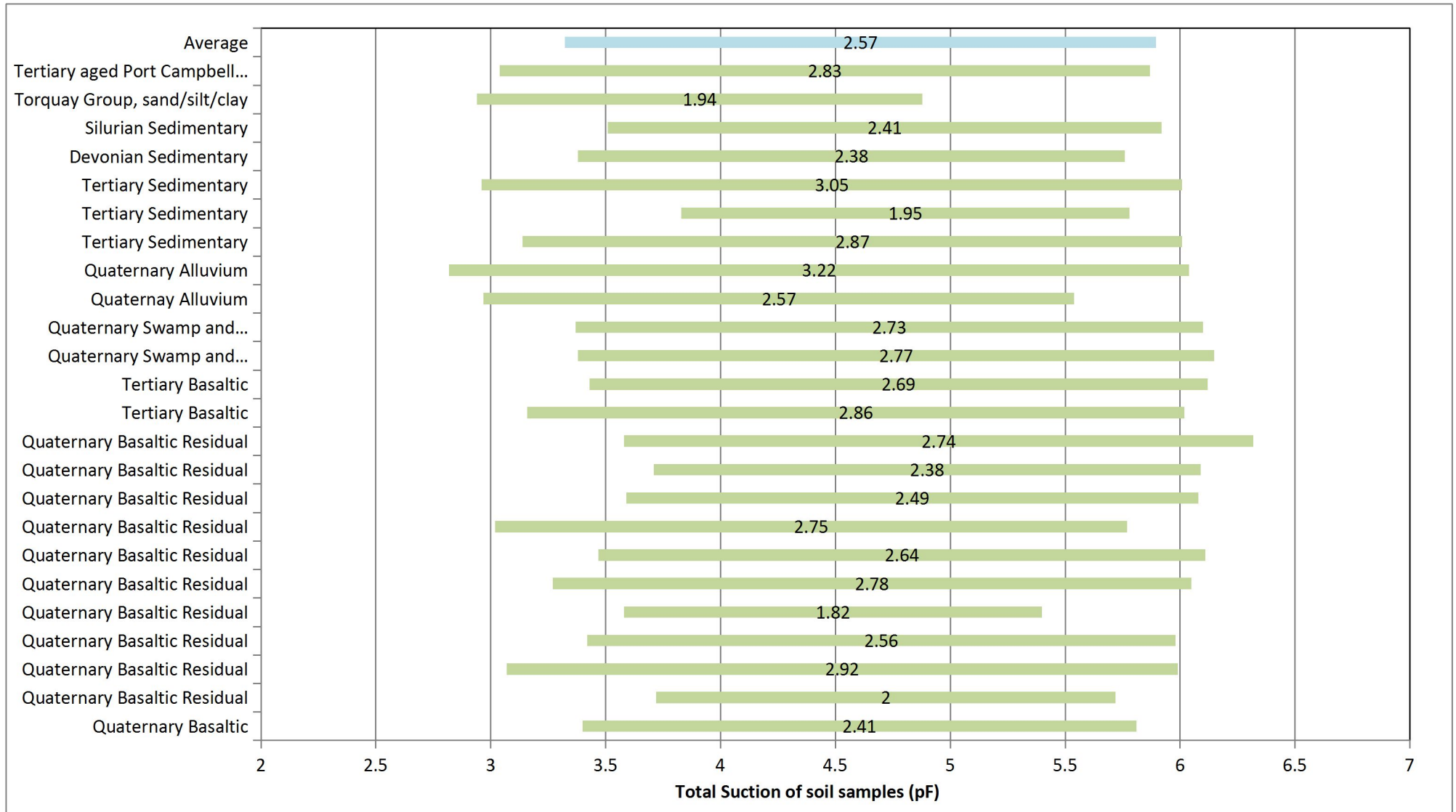


Figure.74 Variation of suction in the shrink-swelling test

Table.12 Statistics of the soil suction change from shrink-swelling test

	Minimum, u_{\min} (pF)	Maximum, u_{\max} (pF)	Mean, \bar{u} (pF)	Standard deviation, σ	Coefficient of Variation, CV (%)	Variation (%)
Soil suction after swelling test (u_{swell})	2.82	3.83	3.36	0.29	8.6	35.82
Soil suction after air dry (u_{dry})	4.88	6.32	5.90	0.30	5.1	29.51
The range of soil suction change (Δu)	1.83	3.22	2.57	0.36	14.2	76.44

Note: variation = $\frac{u_{\max} - u_{\min}}{u_{\min}} \times 100\%$ and CV = $\frac{\sigma}{\bar{u}} \times 100\%$ (i.e., at water contents near saturation)

The values of total suction at the end of swelling after a minimum of ten days of testing range from 2.82 (65 kPa) to 3.83 pF (660 kPa) with a mean value of 3.36 pF (224 kPa), larger than the value of 2.2-2.5 pF (16-31 kPa) reported in [Fredlund & Rahardjo \(1993\)](#). This indicates that the osmotic suction of Melbourne expansive soils may range from 50 kPa to 630 kPa. After air drying for a period of at least 10 days, the suction of the core shrinkage specimens was measured by using a Decagon WP4 Dewpoint Potentiometer. The total suctions at the shrinkage limit vary between 4.88 to 6.32 pF with an average of 5.9 pF, which is consistent with the total suction value of 6 pF (air dry) reported in Lytton, R. L., (1994). From [Table 12](#), it can be seen that during the shrink-swell testing, the range of soil suction change Δu from effective saturation to air dry vary between 1.83 pF to 3.22 pF, with a mean value of 2.57 pF. This suggests that the shrink swell index might be overestimated by approximately 40% and denominator in Equation 1 should be 2.5 rather than 1.8. The results of the core shrinkage test on the initially saturated reconstituted homogeneous Maryland clay sample shows that 80% of the recorded volume change occurred tween total suction of 2.5 pF and 4.5 pF (an interval of 2 pF) over the 2.3 -5.6 pF (effective saturation to air dry) range ([Figure 74](#)). This suggests that a value of 1.8 pF is not unseasonable although a value of 2 may be more appropriate and less conservative.

4.4 Atterberg limit tests

According to Australian standard 2870 ([2011](#)), the soil shrinkage index could be derived from three methods, either from Visual-tactile method or laboratory tests for soil reactivity, or correlations between shrinkage index (I_{pt}) and other clay index tests for soil types ([AS2870, 2011](#)). Atterberg limits test is one of the clay index tests suggested by the Standard.

Table.13 Results of Atterberg limit tests

NO.	Location	Lithology	I_{ss} (%/pF)	LL (%)	PL (%)	PI (%)	LS (%)
1	Brookfield	Quaternary Basalt	1.47	39.94	19.43	20.51	11.20
2	Point Cook	Quaternary Basalt	6.16	65.26	28.51	36.75	20.20
3	Mernda	Quaternary Basalt	2.70	48.20	24.00	24.20	14.00
4	Craigieburn	Quaternary Basalt	3.55	58.61	22.18	36.43	17.10
5	Point cook	Quaternary Basalt	2.83	72.70	25.10	47.60	19.90
6	South Morang	Quaternary Basalt	5.73	100.00	37.02	62.98	25.33
7	Truganina	Quaternary Basalt	7.15	122.00	42.00	80.00	19.60
8	Tullamarine-2	Quaternary Basalt	3.46	38.92	19.27	19.65	12.00
9	Glenroy -3	Quaternary Basalt	2.87	40.65	20.70	19.95	18.00
10	Horsham-1	Quaternary Basalt	2.22	37.73	17.24	20.49	13.40
11	Tullamarine-4	Quaternary Basalt	2.88	36.09	15.46	20.63	11.20
12	Tullamarine-3	Quaternary Basalt	2.56	51.52	30.18	21.34	13.20
13	Horsham-2	Quaternary Basalt	2.57	42.43	20.02	22.41	15.20
14	Glenroy -1	Quaternary Basalt	4.40	49.00	26.18	22.82	20.00
15	Horsham-3	Quaternary Basalt	1.69	41.75	17.21	24.54	15.00
16	Horsham-4	Quaternary Basalt	2.32	46.68	19.48	27.20	15.40
17	Glenroy -2	Quaternary Basalt	3.04	50.77	22.60	28.17	20.40
18	Wollert	Quaternary Basalt	2.95	47.71	15.01	32.70	12.80
19	Williamstown-3	Quaternary Basalt	7.02	74.69	34.87	39.82	24.00
20	Epping	Quaternary Basalt	4.30	73.30	25.80	47.50	18.80
21	Braybrook	Quaternary Basalt	4.53	80.75	18.02	62.73	22.10

Table13 (CONT) Results of Atterberg limit tests

NO.	Location	Lithology	I_{ss} (%/pF)	LL (%)	PL (%)	PI (%)	LS (%)
22	Williamstown-1	Quaternary Basalt	5.96	68.87	31.95	36.92	19.20
23	Williamstown-2	Quaternary Basalt	6.32	84.39	26.62	57.77	24.20
24	Armstrong Creek	Quaternary Basaltic Residual	5.05	47.08	15.81	31.27	13.60
25	Point cook	Quaternary Basaltic Residual	1.59	66.29	18.66	47.63	18.30
26	Tarneit	Quaternary Basaltic Residual	6.96	75.79	24.51	51.28	20.90
27	Wollert	Quaternary Basaltic Residual	6.88	70.28	21.74	48.54	18.40
28	South morang	Quaternary Basaltic Residual	6.16	69.74	19.72	50.02	19.50
29	Diggers rest	Quaternary Basaltic Residual	6.77	80.52	22.37	58.15	23.20
30	Warragul	Tertiary Basaltic	1.08	56.34	31.91	24.43	9.80
31	Warragul	Tertiary Basaltic	3.28	49.99	29.74	20.25	12.50
32	Clyde north	Tertiary Basaltic Residual Soils	1.87	63.91	21.28	42.63	11.10
33	Clyde north	Quaternary Swamp and Lagoon Deposits	4.21	56.92	17.42	39.50	16.00
34	Cranbourne north	Quaternary Swamp and Lagoon Deposits	2.42	64.17	20.57	43.60	16.10
35	Officer	Quaternary Aged Alluvium Deposits	1.91	36.01	12.82	23.19	17.90
36	Vermont-2	Silurian Sedimentary	1.57	57.60	24.91	32.69	10.40
37	Doreen	Silurian Sedimentary	4.02	61.10	25.70	35.40	15.20
38	Vermont-1	Silurian Sedimentary	1.75	54.79	19.27	35.52	12.00
39	Macleod	Silurian Sedimentary Clays	2.51	65.93	25.74	40.19	14.59
40	Hastings	Tertiary Aged Sedimentary Deposits	2.55	54.96	27.16	27.80	12.70
41	Cranbourne North	Tertiary Aged Sedimentary deposits	2.11	43.34	14.89	28.45	8.30
42	Torquay	Torquay Group, sand/silt/clay deposits	3.06	69.80	23.50	46.30	15.50
43	Lyndhurst	Quaternary Alluvium	3.23	41.53	16.86	24.67	13.20
44	Lyndhurst	Quaternary Alluvium	3.76	48.88	18.65	30.23	14.60

A total of 41 soil samples were used for the Atterberg tests. The samples were obtained from a range of suburbs across Melbourne area. Table 13 summarises the results the Atterberg limit tests results which include:

- Liquid limit test results
- Plastic limit test results
- Plastic index
- Linear shrinkage test results

The results are also plotted on the plasticity chart as shown in Figure 75. It can be seen that most samples are above the A-line, and are classified as low to high plasticity clay.

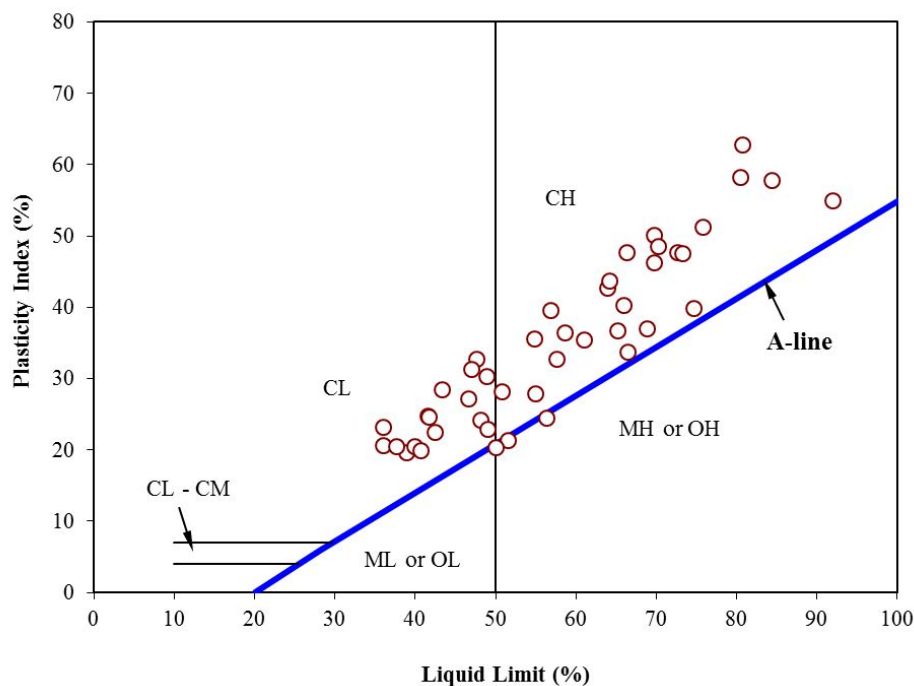


Figure.75 Plasticity chart

4.5 Correlation between the shrink-swell index and traditional soil indices

For most engineering purposes, the properties of soil can be defined by index tests such as plastic index, linear shrinkage or liquid limit. Several studies have been carried out in an attempt to establish possible relationships between these traditional soil indices and the instability indices. Mitchell and Avalle (1984) found linear shrinkage to be a reasonable

indicator of soil reactivity. Cameron suggests that that greater success would be achieved if a correlation were confined to a single soil type. A study undertaken by [Delaney, Li and Fityus \(2005\)](#) indicates that in Newcastle there is a poor correlation between traditional soil indices and I_{ss} .

A number of correlations were attempted in this study between the shrink-swell test and other soil tests including linear shrinkage, plastic index and liquid limit. The results are based the laboratory tests on 44 different soil samples tests; collected from 26 suburbs in Melbourne.

4.5.1 Correlation between the shrink- swell index and liquid limit

To determine if a correlation exists, the values of the shrink-swelling indices were plotted against liquid limit in [Figure 76](#) using Excel spreadsheet. The equation of the fitted trend line and R^2 values are:

$$I_{ss} = 0.0667 \text{ LL} - 0.286, \quad R^2 = 0.434$$

For a correlation to be considered as a reliable mean for estimating the shrink swell index of a soil, R^2 of the trend line needs to exceed 0.8. It is apparent that there is a poor correlation between liquid limit and I_{ss} since the strength of this correlation is only 43.4% (i.e. $R^2 = 0.43$).

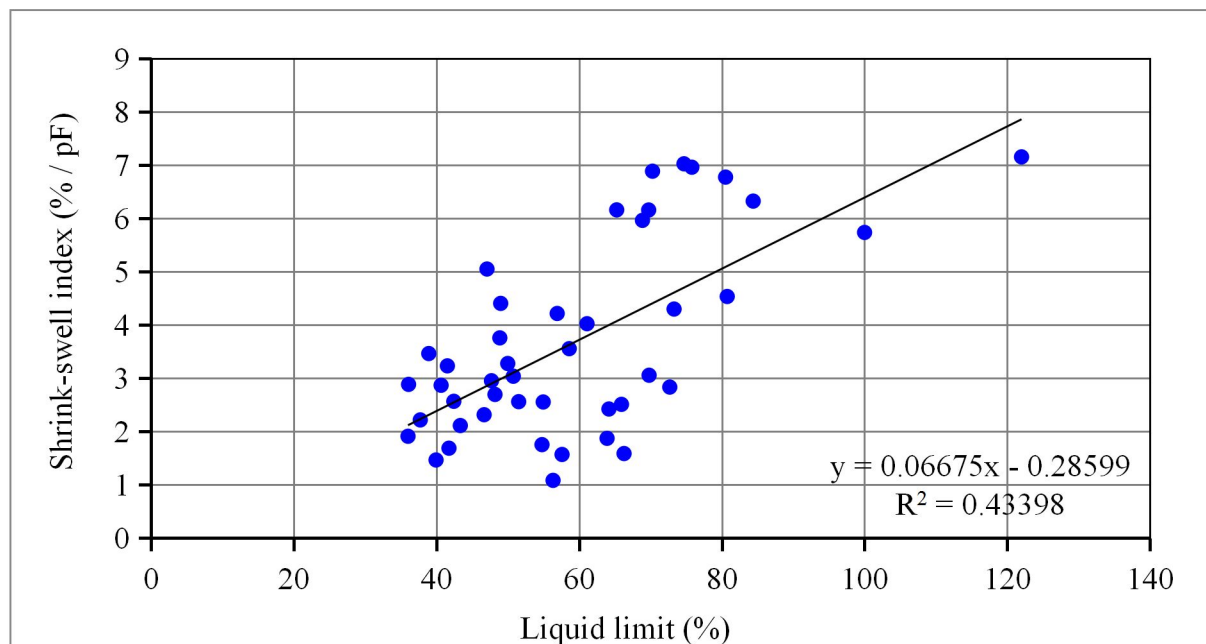


Figure.76 Shrink-swell index vs liquid limit (a total of 44 soil samples)

Based on the results of the laboratory testing on soils samples collected from twenty field sites in the Newcastle-Hunter region, [Delaney, Li and Fityus \(2005\)](#) found that traditional index tests were of little general value in assessing the reactivity of expansive soil unless the correlation was confined to a particular soil type. Thus, a number of correlations which were confined to a particular soil type were attempted in this study.

In this study, Basaltic and sedimentary soils dominate the soil data. The Quaternary basaltic soil and Quaternary Basaltic residual soil comprise about 62% of the samples collected/tested, and 67% when combined with Tertiary Basaltic . Approximately 33% of the soil data can be grouped into other broad categories, i.e. Sedimentary soils.

4.5.1.1 Basaltic soil

[Figure 77](#) shows the relationship between I_{ss} and liquid limit for all Basaltic soil samples. While there is indeed an obvious trend between the LL and I_{ss} , the correlation is very scattered.

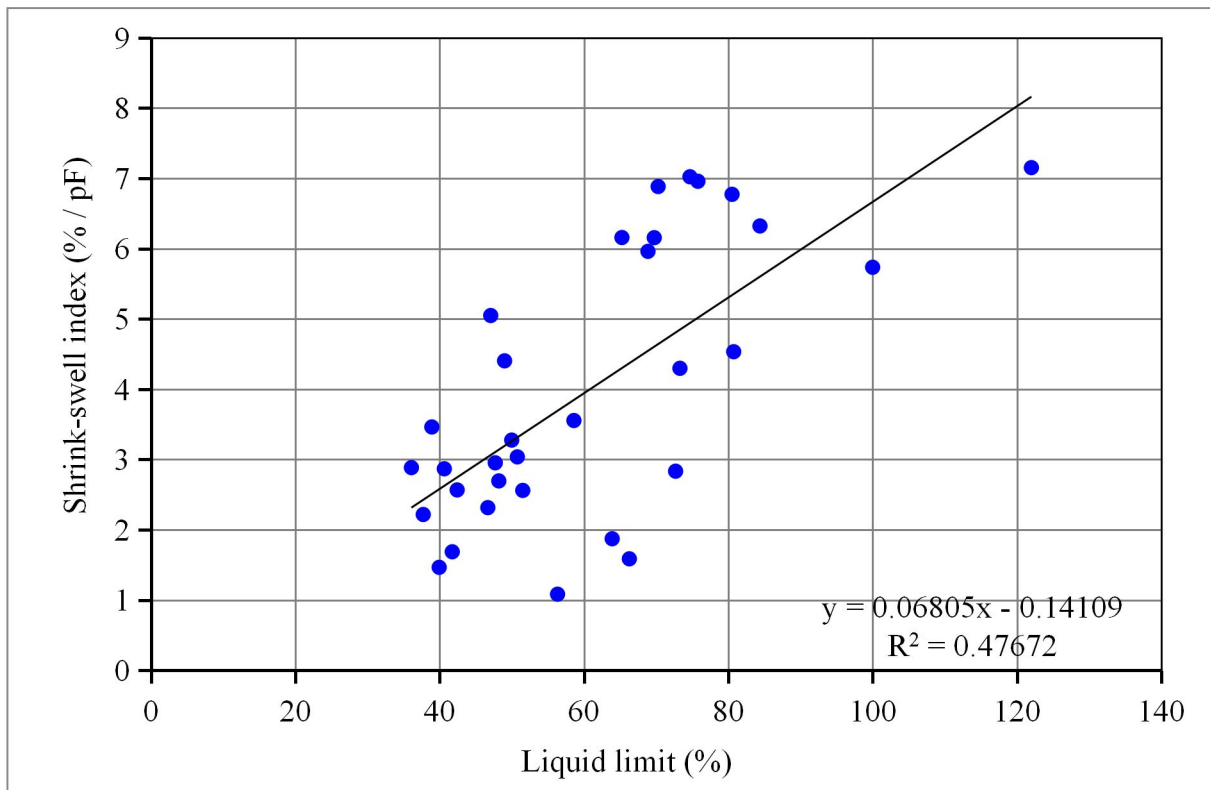


Figure.77 Shrink-swell index vs liquid limit (all Basaltic soil samples)

The basaltic soils samples tested in this study can be categorized into three groups: Tertiary basaltic soil, Quaternary basaltic residual soil and Quaternary basaltic soil. Further attempts are made to determine whether there is a correlation between I_{ss} and LL for each type of basaltic soil. The results are presented in [Figures 78 to 80](#). There is an improved correlation between I_{ss} and LL for Quaternary basaltic soil ([Figures 78-80](#)) but no correlation of I_{ss} with LL for Tertiary basaltic soil (refer to [Figure 80](#)).

It should be pointed out that one Quaternary basaltic residual soil sample from Point Cook has a very low value of I_{ss} due to high calcium carbonate content. This sample was not included in the analysis (refer to [Figure 79](#)). The calcium carbonate can significantly reduce the soil reactivity. Cameron (1989) reported that the Newer Volcanics clay in Melbourne had an average shrink-swell index of 5.5 %/pF for relatively non-calcareous clays but the index may dropped to approximately 2%/pF for carbonate-rich layers.

After ignoring the data from point cook, a reasonably good correlation can be observed between I_{ss} and LL for Quaternary basaltic residual soil, the equation of the fitted trend line and R^2 values are:

$$y = 0.0576x + 2.4054 \quad R^2 = 0.856$$

The strength of this equation is 85.6% (i.e. $R^2 = 0.856$). According to the strength of this correlation it is deemed a reliable correction between plastic limit and the shrink swell index, however this relationship still cannot be used for all Quaternary basaltic residual soils since only five tests were performed.

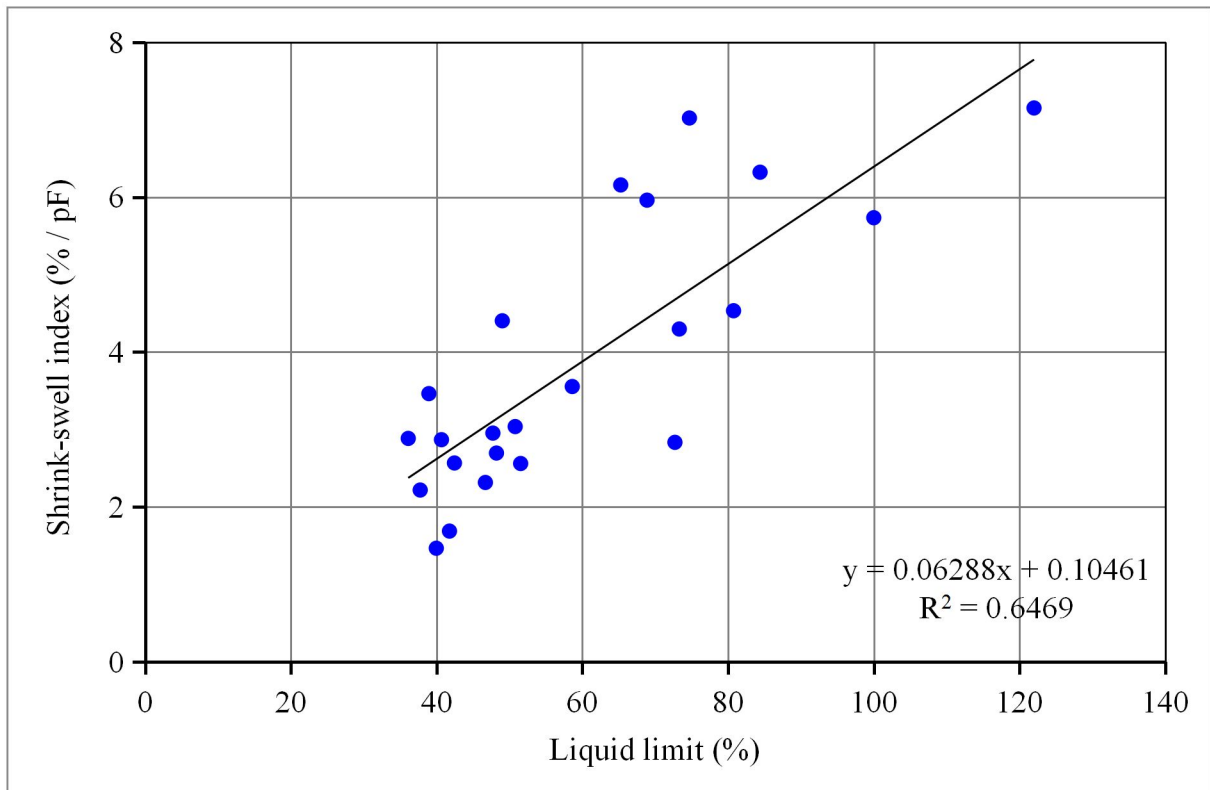


Figure.78 Shrink-swell index vs liquid limit (Quaternary basaltic soil samples)

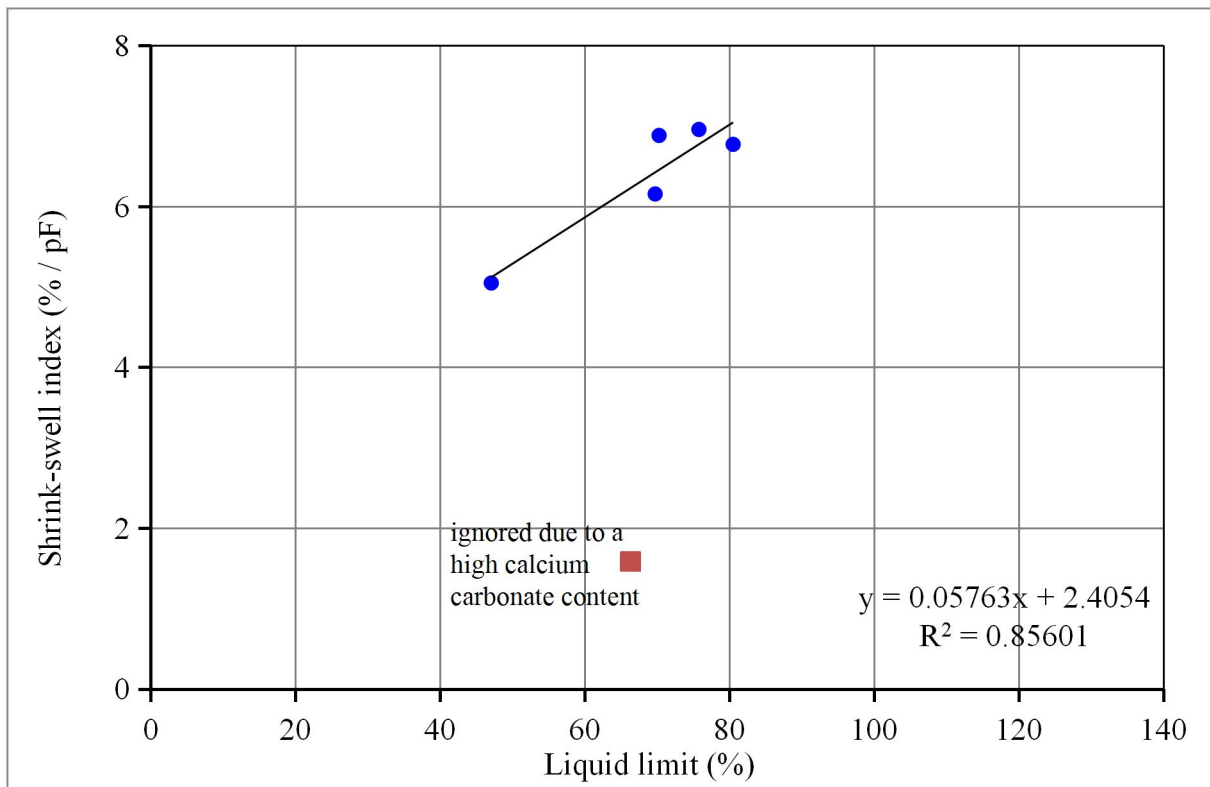


Figure.79 Shrink-swell index vs liquid limit (Quaternary basaltic residual soil samples)

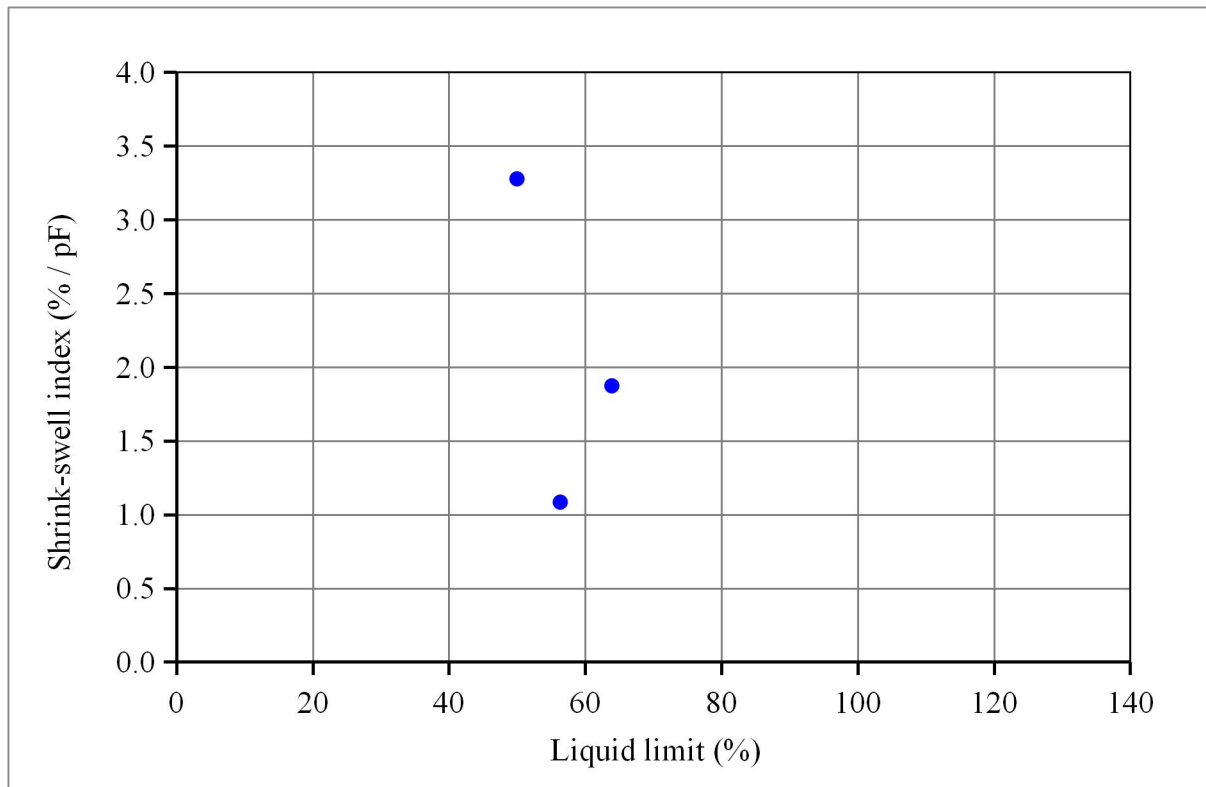


Figure.80 Shrink-swell index vs liquid limit (Tertiary basaltic soil samples)

4.5.1.2 Sedimentary soil

A similar correlation analysis was also conducted on Sedimentary soil. [Figures 81-83](#) show that significant scatter existed in the relationship between the shrink-swell index and liquid limit in sedimentary soil, so that an unambiguous, reliable relationship does not appear to exist. Although there is a reasonably good correlation between I_{ss} and LL for Quaternary alluvium soils, this relationship cannot be applied for all soils since only a limited number of tests were used in the analysis.

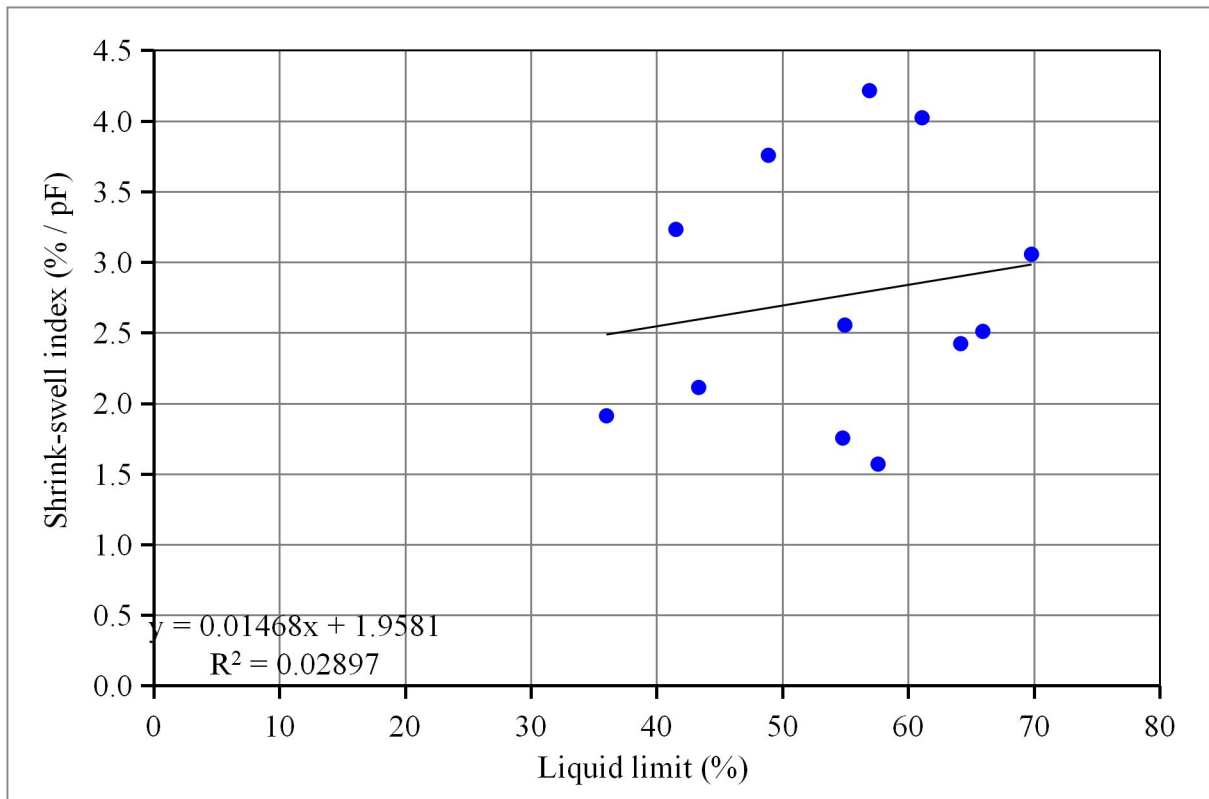


Figure.81 Shrink-swell index vs liquid limit (all sediment soil samples)

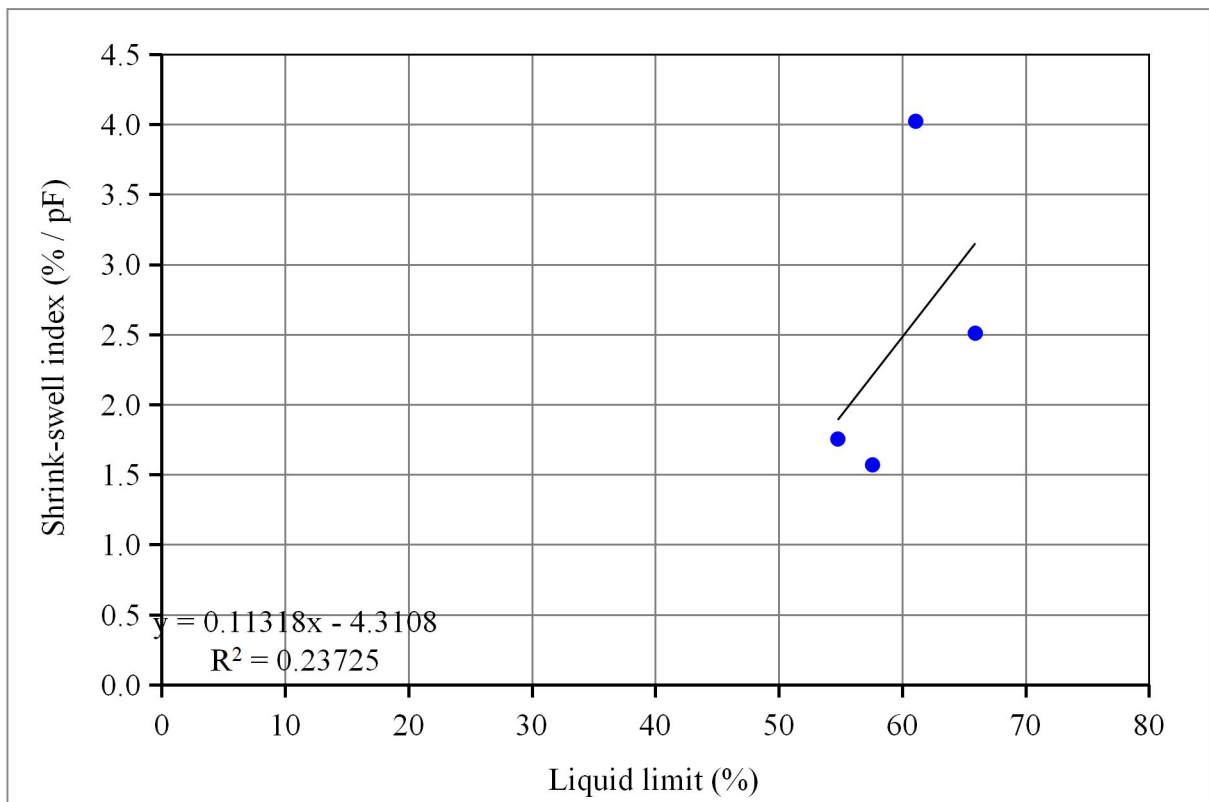


Figure.82 Shrink-swell index vs liquid limit (Silurian sediment soil samples)

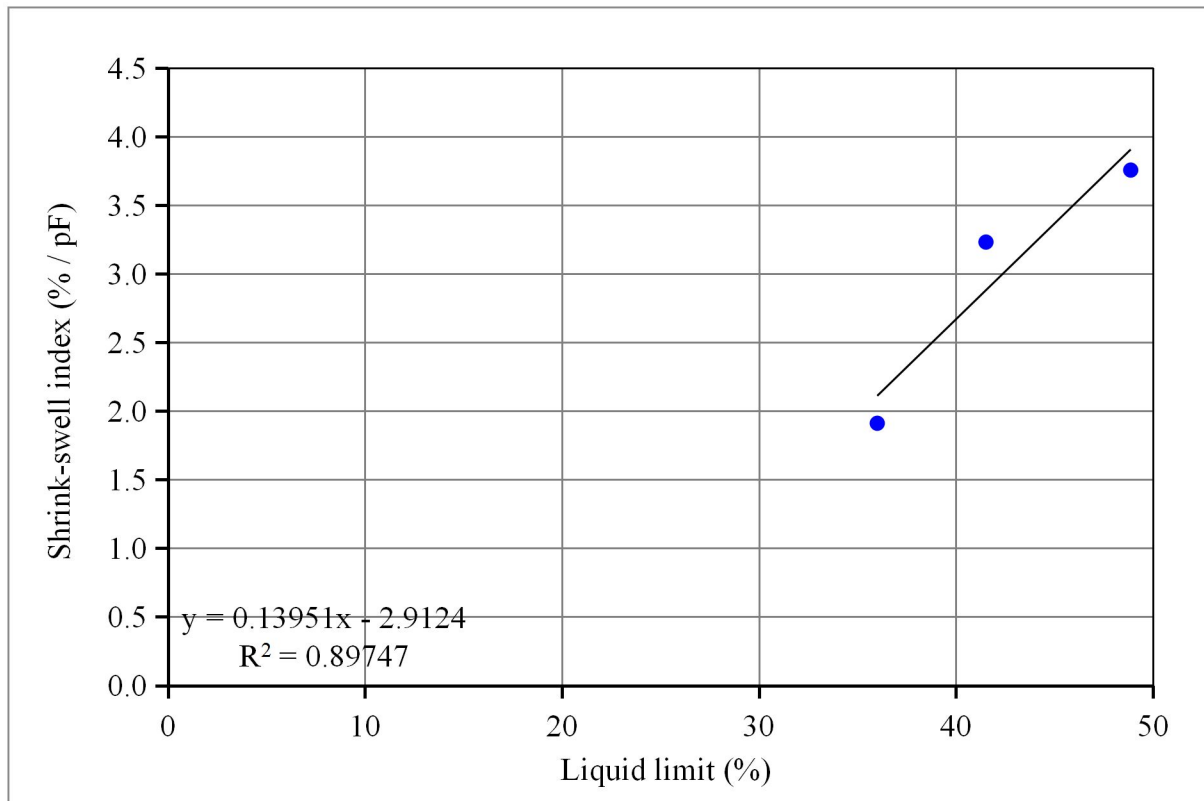


Figure.83 Shrink-swell index vs liquid limit (Quaternary alluvium soil samples)

4.5.2 Correlation between the shrink swell index and plastic limit

The shrink-swell index plotted against plastic limit in [Figure 84](#) indicates a large spread of results.

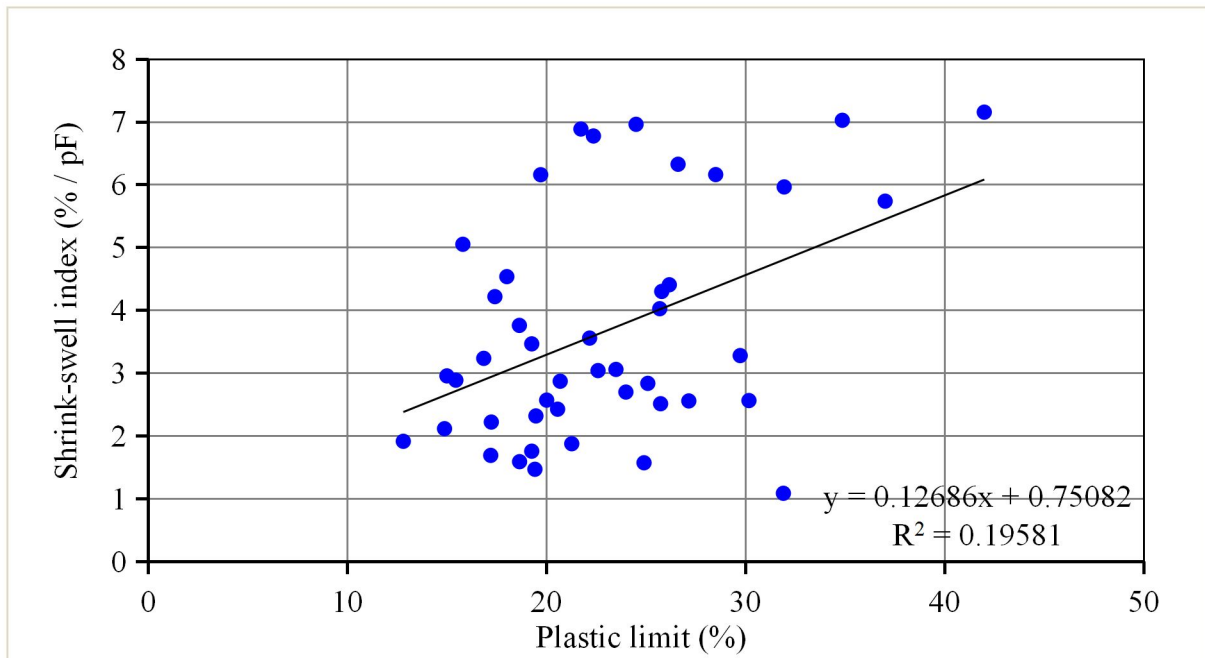


Figure.84 Shrink-swell index vs plastic limit (all soil samples)

To determine if a correlation exists between the shrink-swell index and the plastic limit, the values of the shrink-swelling indices were plotted against plastic limit shown in [Figure 84](#). The equation of the fitted trend line and R^2 values are:

$$I_{ss} = 0.1269PL + 0.7508, \quad R^2 = 0.1958$$

For a correlation to be considered as a reliable mean for estimating the shrink swell index of a soil, R^2 of the trend line needs to exceed 0.8. It is apparent that there is a very weak correlation between plastic limit and I_{ss} since the strength of this correlation is only 19.58% (i.e. $R^2 = 0.1958$).

Although the results of the laboratory testing found that traditional index tests were of little general value in assessing the reactivity of expansive soil, the correlation may be confined to a particular soil type. Thus, a number of correlations which were confined to a particular soil type were attempted in this study. The following investigations were carried out for 2 categories and 5 different types of soil that are Tertiary basaltic soil, Quaternary basaltic residual soil, Quaternary basaltic soil, Quaternary alluvium soils and Silurian sediment soil.

4.5.2.1 Basaltic soil

Figure 85 shows the relationship between I_{ss} and PL of all Basaltic soil samples. While there is indeed an obvious trend between the PL and I_{ss} , the correlation is very scattered.

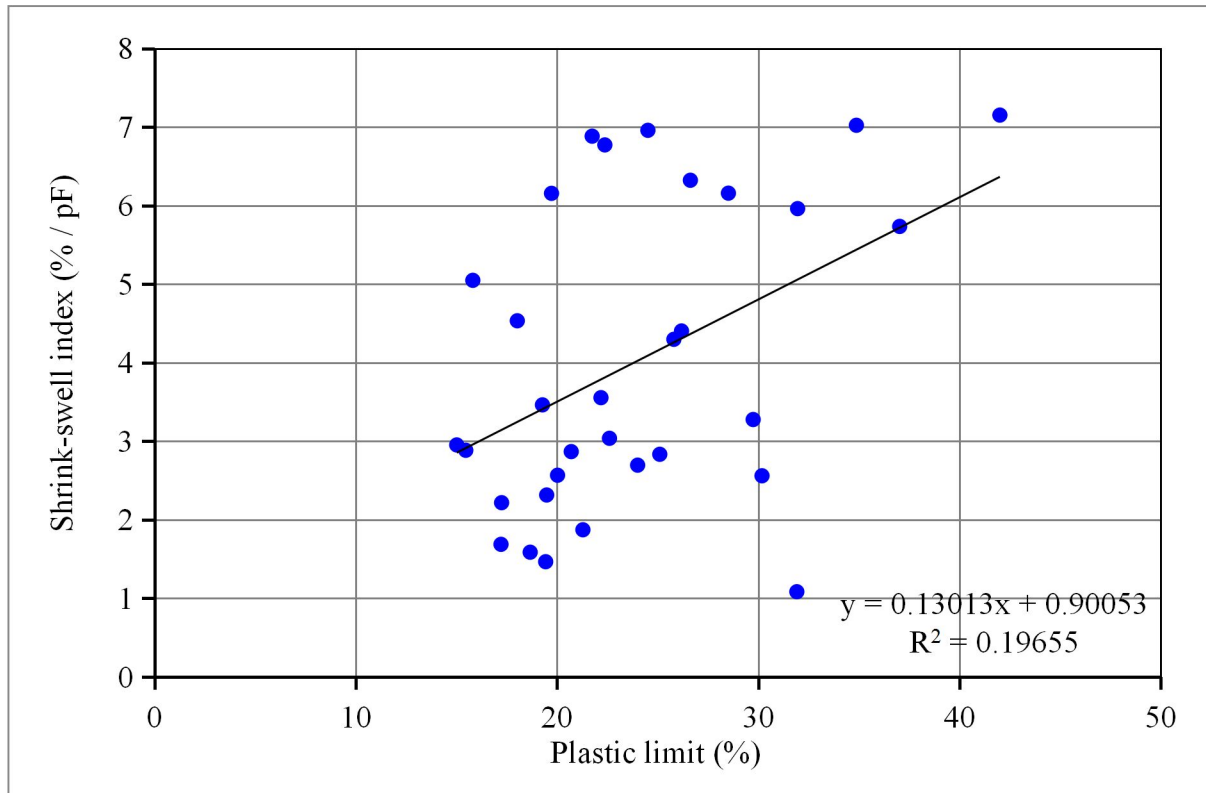


Figure.85 Shrink-swell index vs plastic limit (all Basaltic soil samples)

The basaltic soils samples tested in this study can be categorized into three groups: Tertiary basaltic soil, Quaternary basaltic residual soil and Quaternary basaltic soil. Further attempts are made to determine whether there is a correlation between I_{ss} and PL for each type of basaltic soil. The results are presented in Figures 86 to 88. There is an improved correlation between I_{ss} and PL for Quaternary basaltic soil (shown in Figure 86) and no correlation of I_{ss} with PL for Tertiary basaltic soil as shown in Figure 88.

It should be pointed out that one Quaternary basaltic residual soil sample from Point Cook has a very low value of I_{ss} due to high calcium carbonate content. This sample was not included in the analysis (refer to Figure 87). After ignoring the data from Point Cook, a reasonably good correlation can be observed between I_{ss} and PL for Quaternary basaltic residual soil, the equation of the fitted trend line and R^2 values are:

$$y = 0.8145x - 6.9907 \quad R^2 = 0.9259$$

The strength of this equation is 92.59% (i.e. $R^2 = 0.9259$). According to the strength of this correlation it is deemed a reliable correction between plastic limit and the shrink swell index, however due to a limited number of samples tested it may not appropriate to apply this relationship to all Quaternary basaltic residual soils.

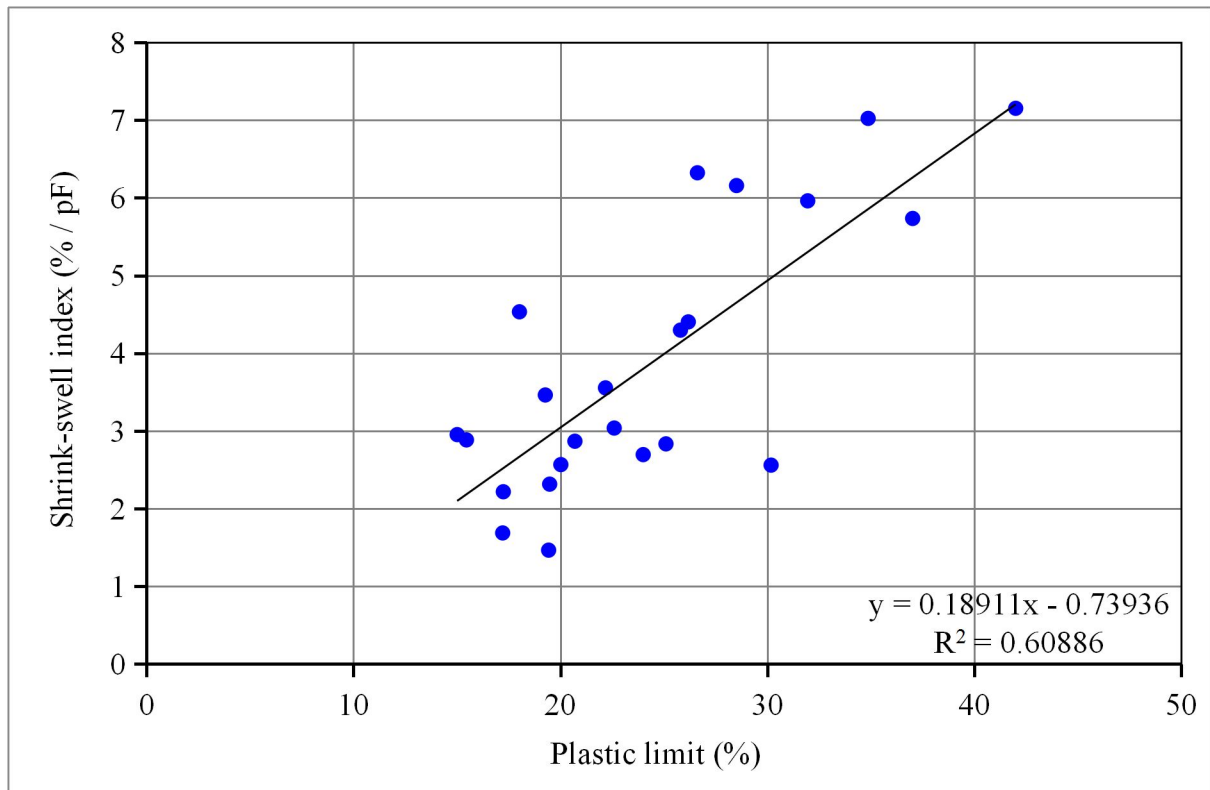


Figure.86 Shrink-swell index vs plastic limit (Quaternary basaltic soil samples)

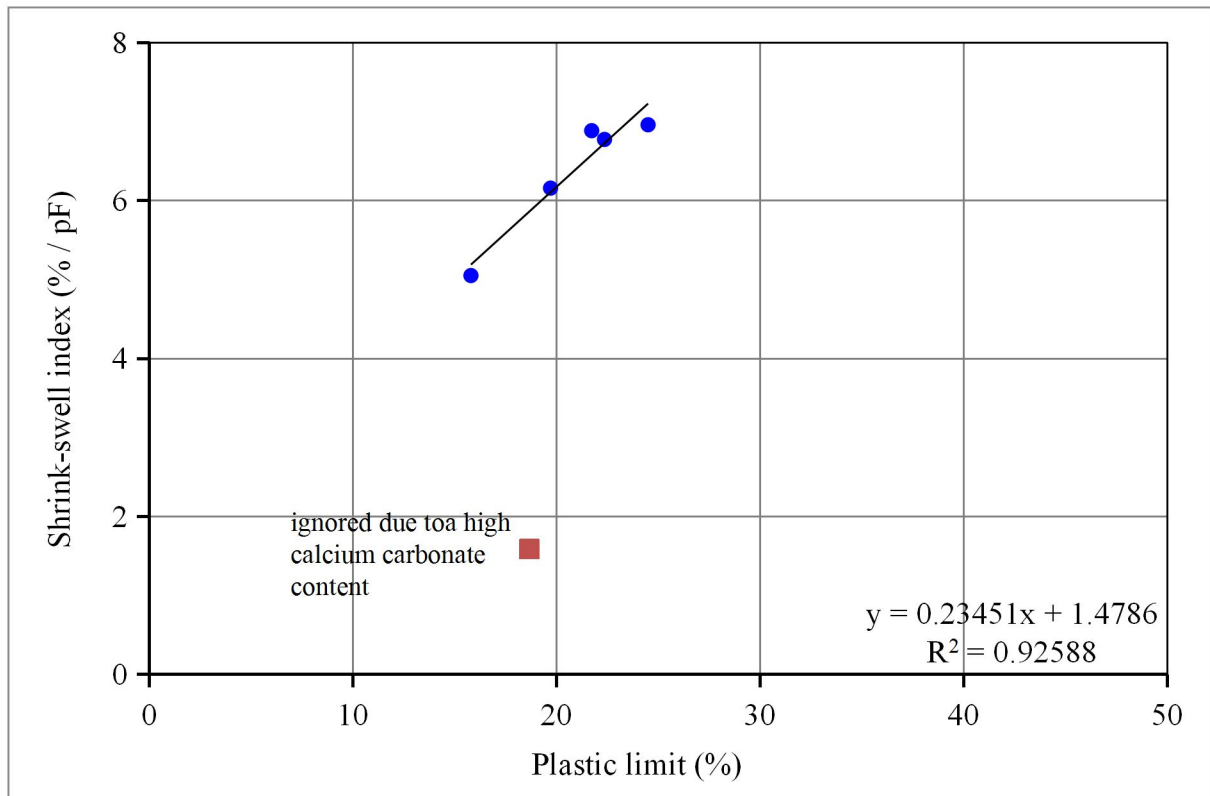


Figure.87 Shrink-swell index vs plastic limit (Quaternary basaltic residual soil samples)

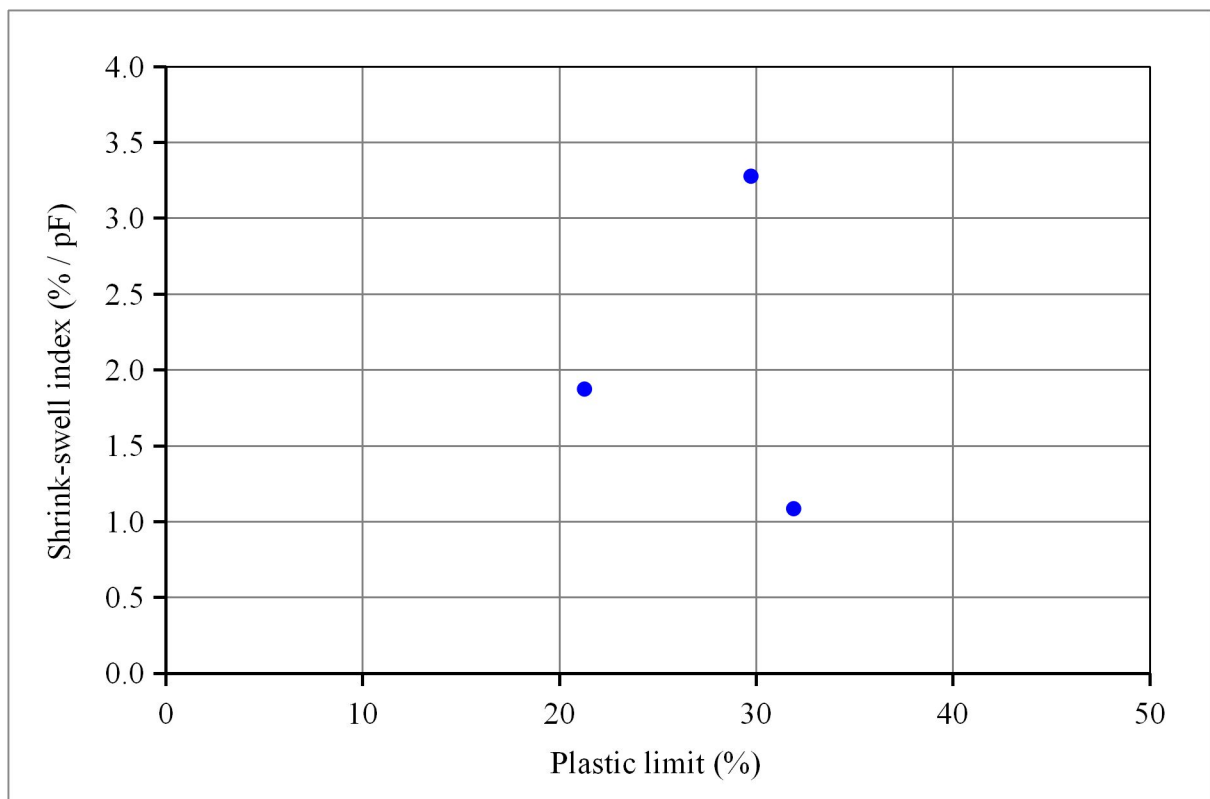


Figure.88 Shrink-swell index vs plastic limit (Tertiary basaltic soil samples)

4.5.2.2 Sedimentary soil

A similar correlation analysis was also conducted on Sedimentary soil. Figures 89-91 show that significant scatter existed in the relationship between the shrink-swell index and plastic limit, so that an unambiguous, reliable relationship does not appear to exist. Although there is a reasonably good correlation can be observed between I_{ss} and PL for Quaternary alluvium soils, it may not appropriate to apply this relationship to all soils since only a limited number of samples were considered in the analysis.

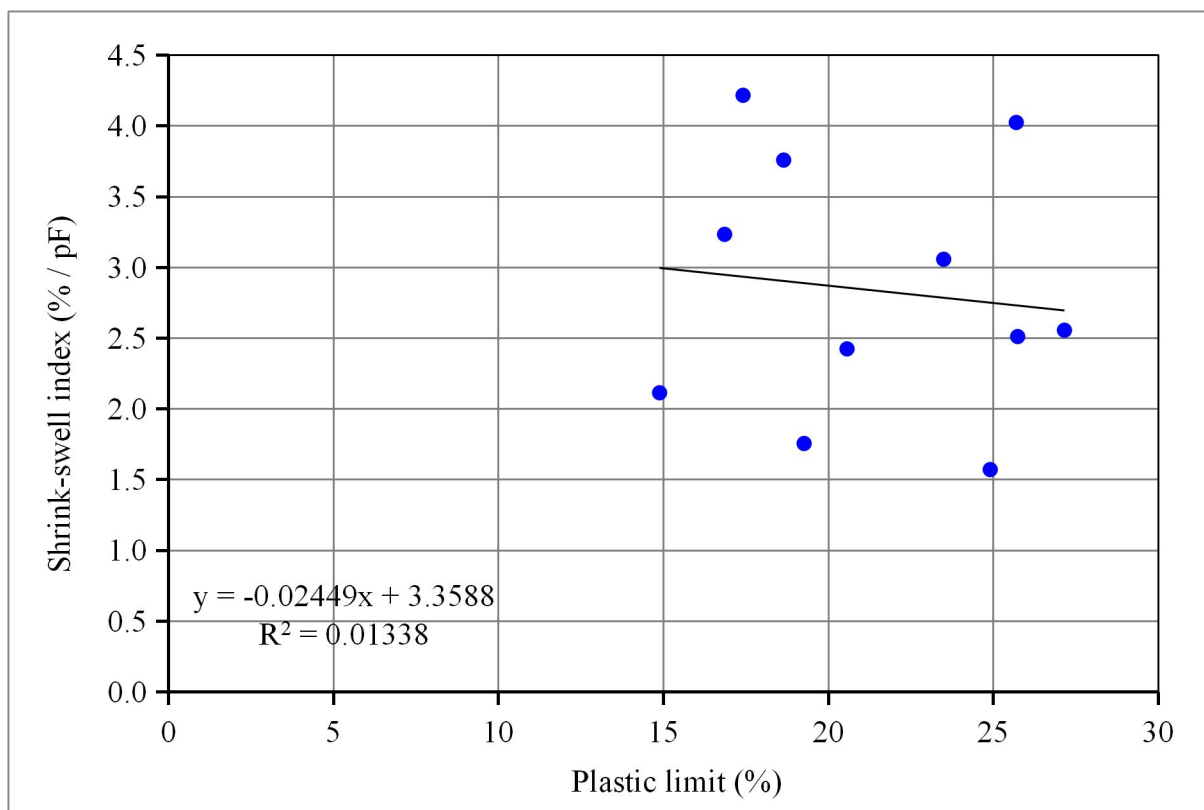


Figure.89 Shrink-swell index vs plastic limit (all sediment soil samples)

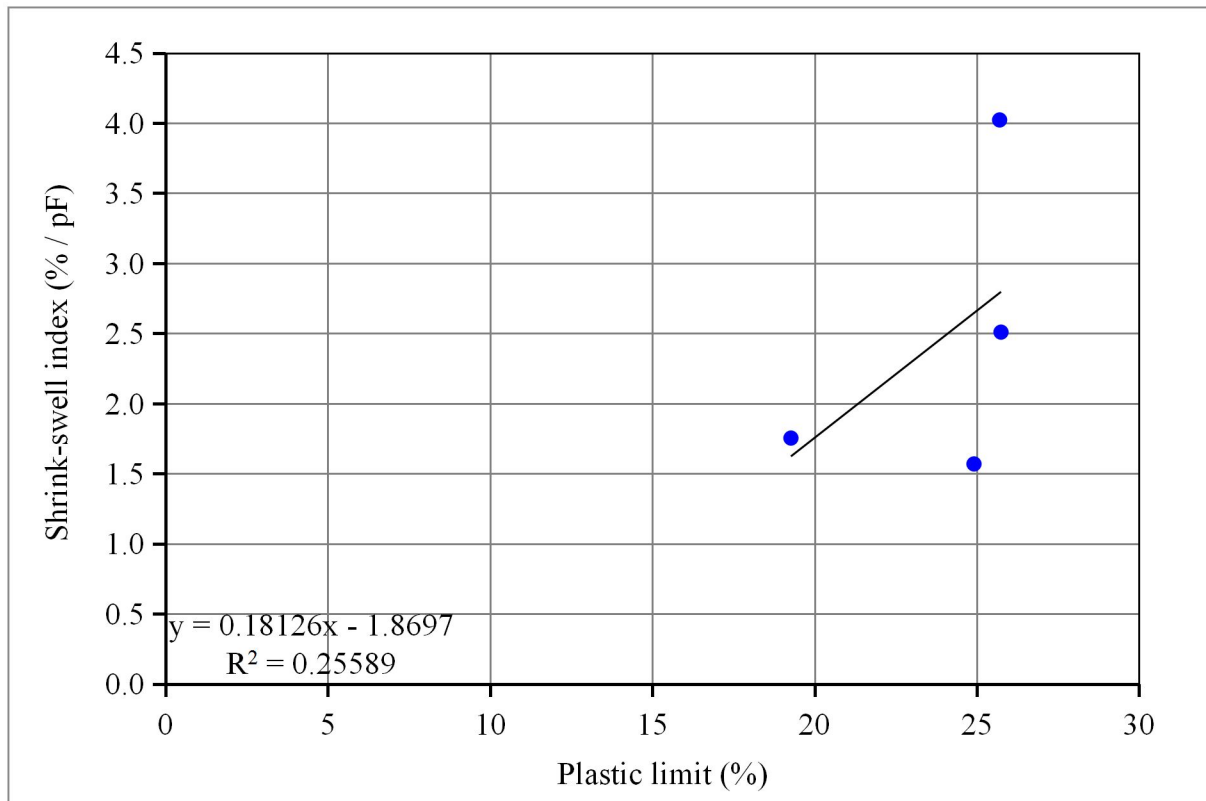


Figure.90 Shrink-swell index vs plastic limit (Silurian sediment soil samples)

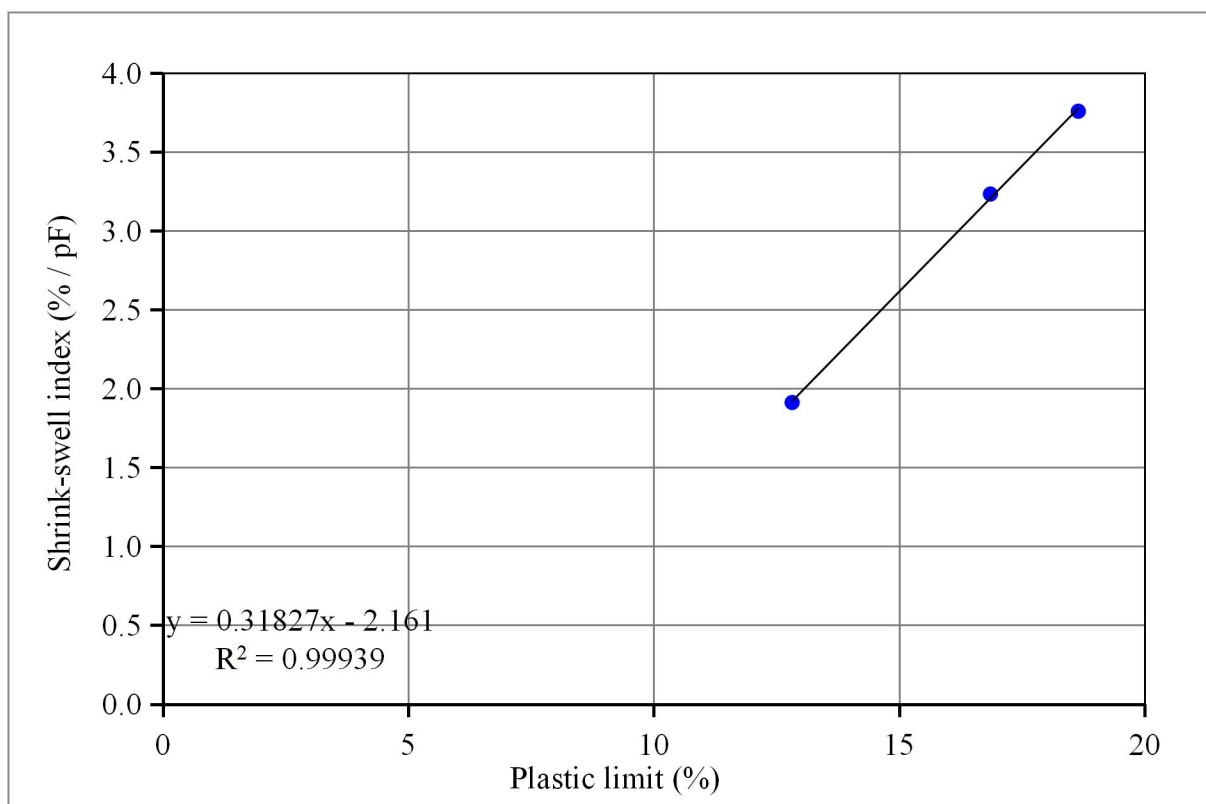


Figure.91 Shrink-swell index vs plastic limit (Quaternary alluvium soil samples)

4.5.3 Correlation between the shrink-swell index and plastic index

The correlation between the shrink swell index and the plastic index for all of the tested is shown [Figure 92](#).

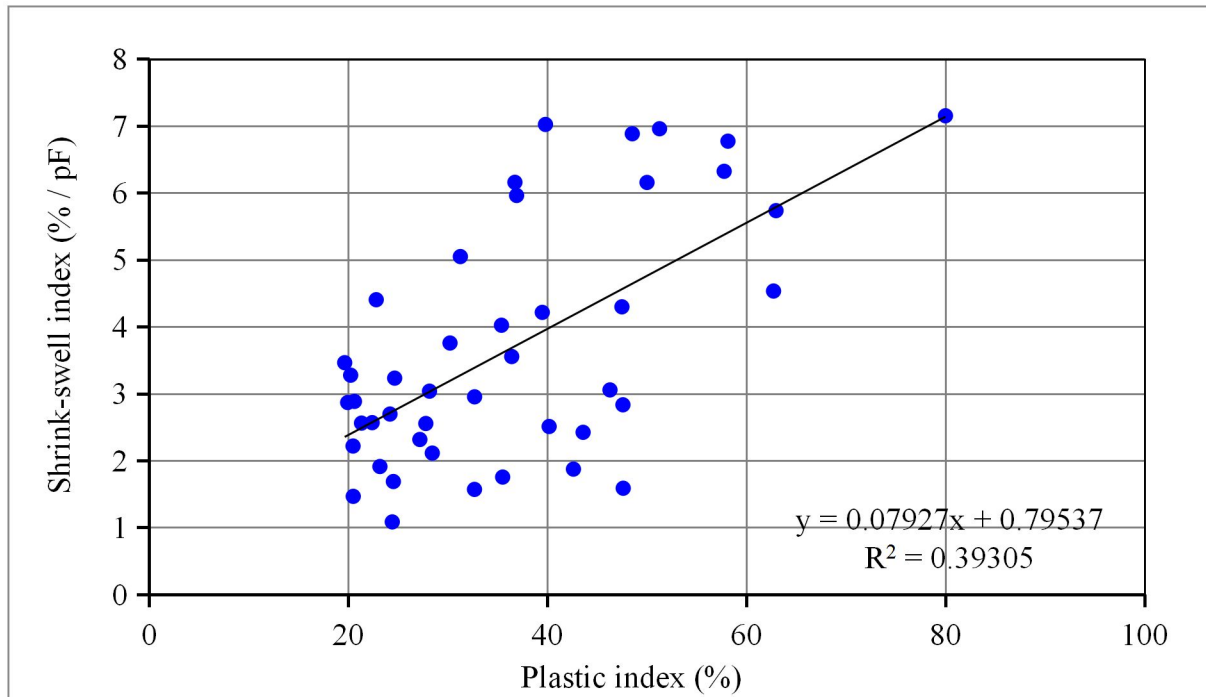


Figure.92 Shrink-swell index vs plastic index (all soil samples)

The last Atterberg limit to be analyzed is the plasticity index. The plasticity index of a soil is the difference between the liquid limit and the plastic limit. To determine if a correlation exists between the I_{ss} and the PI, the values of the shrink-swelling indices were plotted against plastic index as shown in [Figure 92](#). The equation of the fitted trend line and R^2 values are:

$$y = 0.0793x + 0.7954 \quad R^2 = 0.393$$

For a correlation to be considered as a reliable mean for estimating the shrink swell index of a soil, R^2 of the trend line needs to exceed 0.8. It is apparent that there is a poor correlation between PI and I_{ss} since the strength of this correlation is only 39.3% (i.e. $R^2 = 0.393$).

Although the results of the laboratory testing found that traditional index tests were of

little general value in assessing the reactivity of expansive soil, the correlation may be confined to a particular soil type. Thus, a number of correlations which were confined to a particular soil type were attempted in this study. The following investigations are carried out for 2 categories and 5 different types of soils (i.e. Tertiary basaltic soil, Quaternary basaltic residual soil, Quaternary basaltic soil, Quaternary alluvium soils and Silurian sediment soil).

4.5.3.1 Basaltic soil

Figure 93 shows the relationship between I_{ss} and PI of all Basaltic soil samples. While there is indeed an obvious trend between the PI and I_{ss} , the correlation is very scattered.

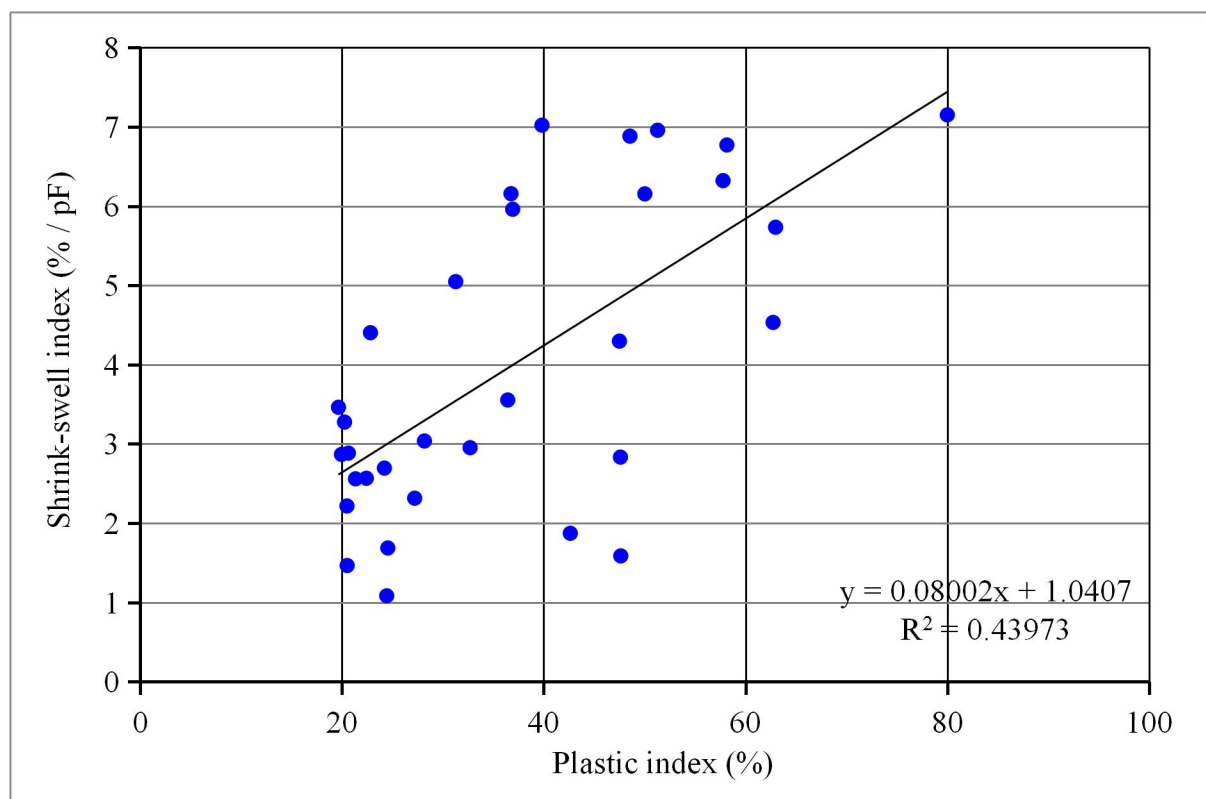


Figure.93 Shrink-swell index vs plastic index (All Basaltic soil samples)

The basaltic soils samples tested in this study can be categorized into three groups: (a) Tertiary basaltic soil, (b) Quaternary basaltic residual soil and (c) Quaternary basaltic soil. Further attempts are made to determine whether there is a correlation between I_{ss} and PI for each type of basaltic soil. The results are presented in Figures 94 to 96. There is an improved correlation between I_{ss} and PI for Quaternary basaltic soil and Quaternary basaltic residual

soil, but no correlation between I_{ss} and PI for Tertiary basaltic soil as shown in [Figure 96](#). It should be pointed out that one Quaternary basaltic residual soil sample from Point Cook has a very low value of I_{ss} due to high calcium carbonate content. This sample was not included in the analysis (refer to [Figure 95](#)).

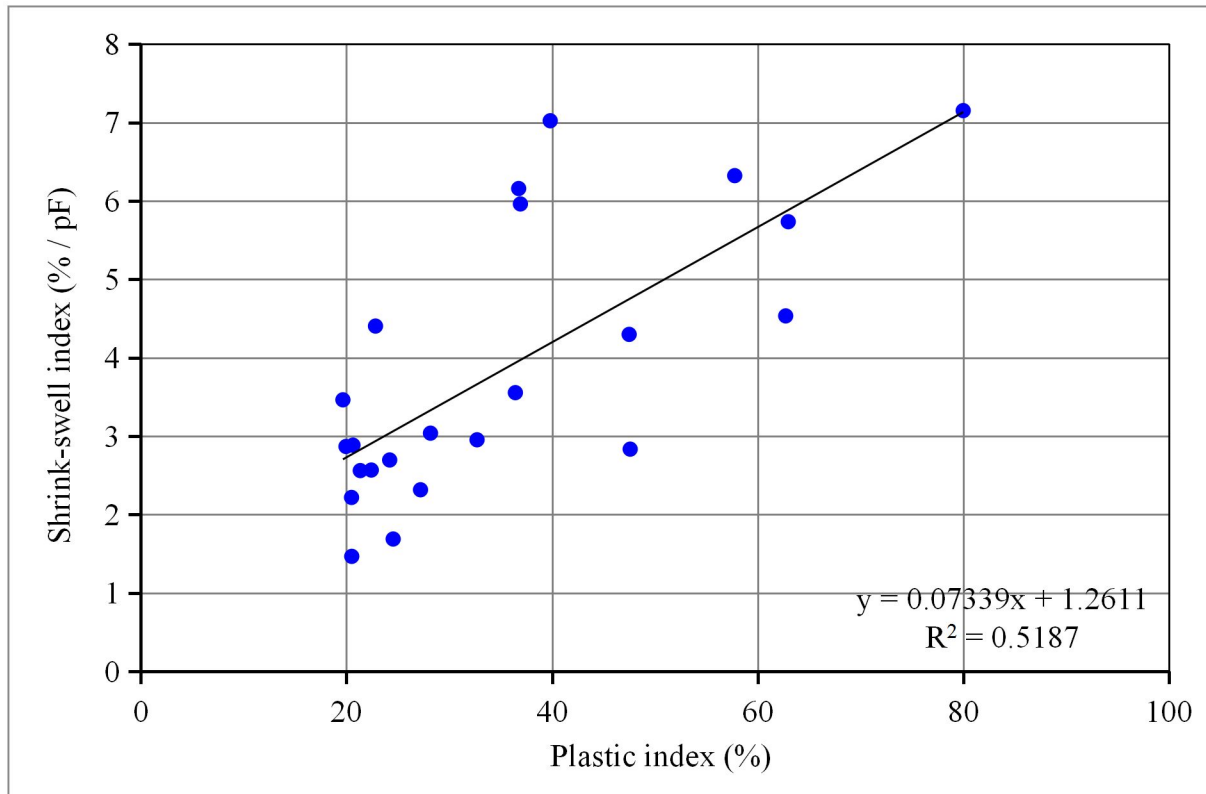


Figure.94 Shrink swell index vs plastic index (Quaternary basaltic soil samples)

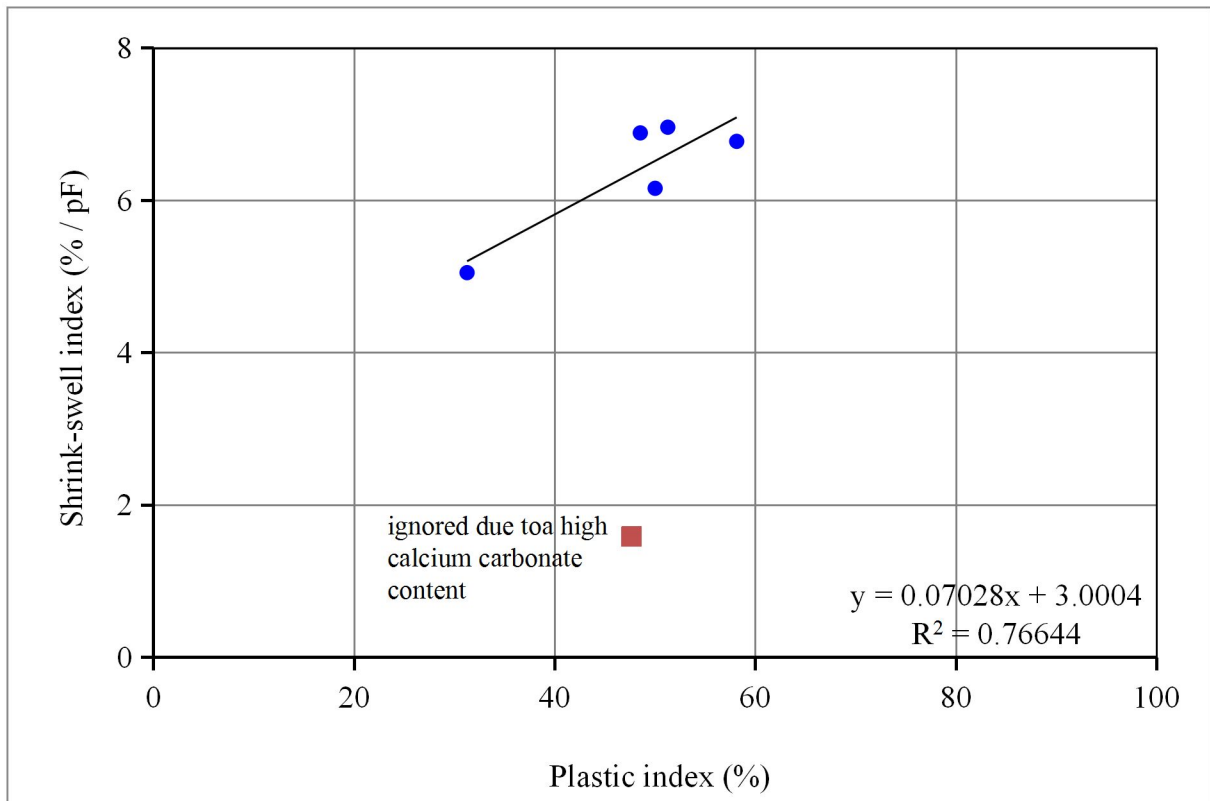


Figure.95 Shrink-swell index vs plastic index (Quaternary basaltic residual soil samples)

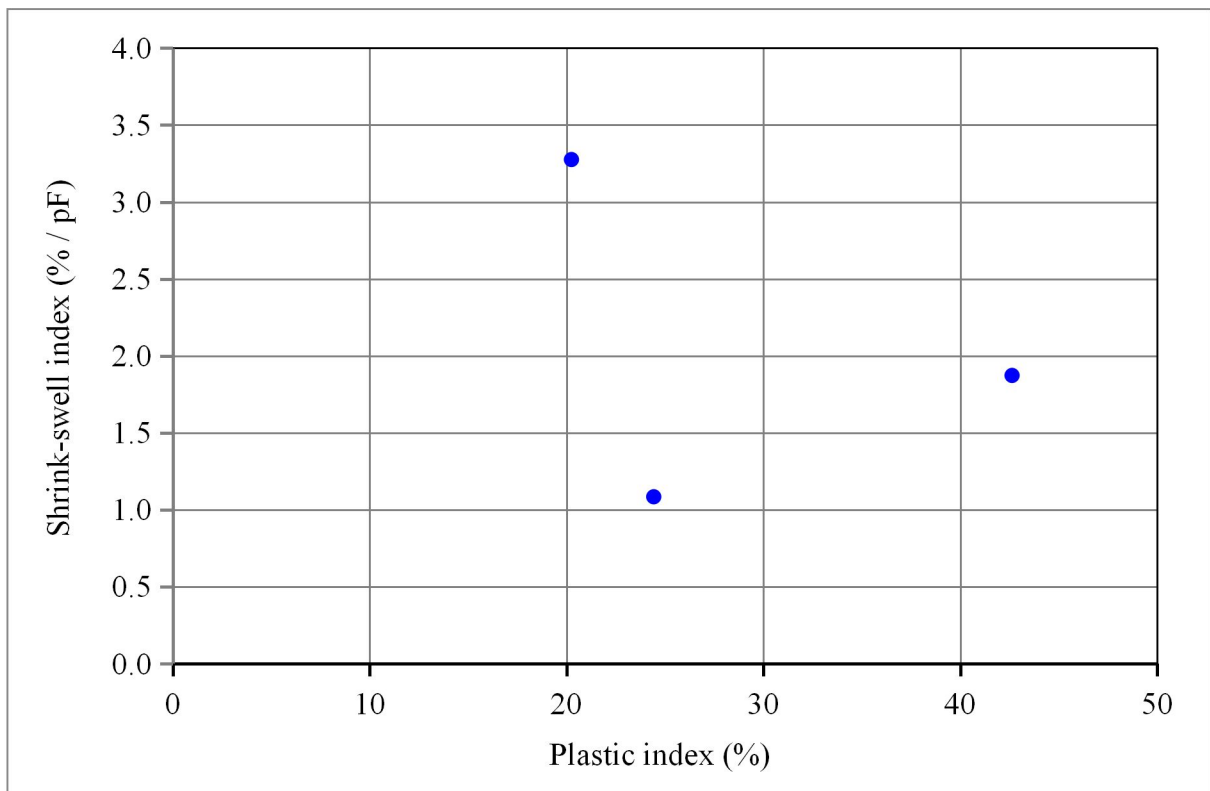


Figure.96 Shrink-swell index vs plastic index (Tertiary basaltic soil samples)

4.5.3.2 Sedimentary soil

A similar correlation analysis was also conducted on Sedimentary soil. Figures 97-99 show that significant scatter existed in the relationship between the shrink-swell index and plastic index, so that an unambiguous and reliable relationship does not appear to exist. Although there is a good correlation between I_{ss} and PI for Quaternary alluvium soils, this relationship cannot be applied to all Quaternary alluvium soils.

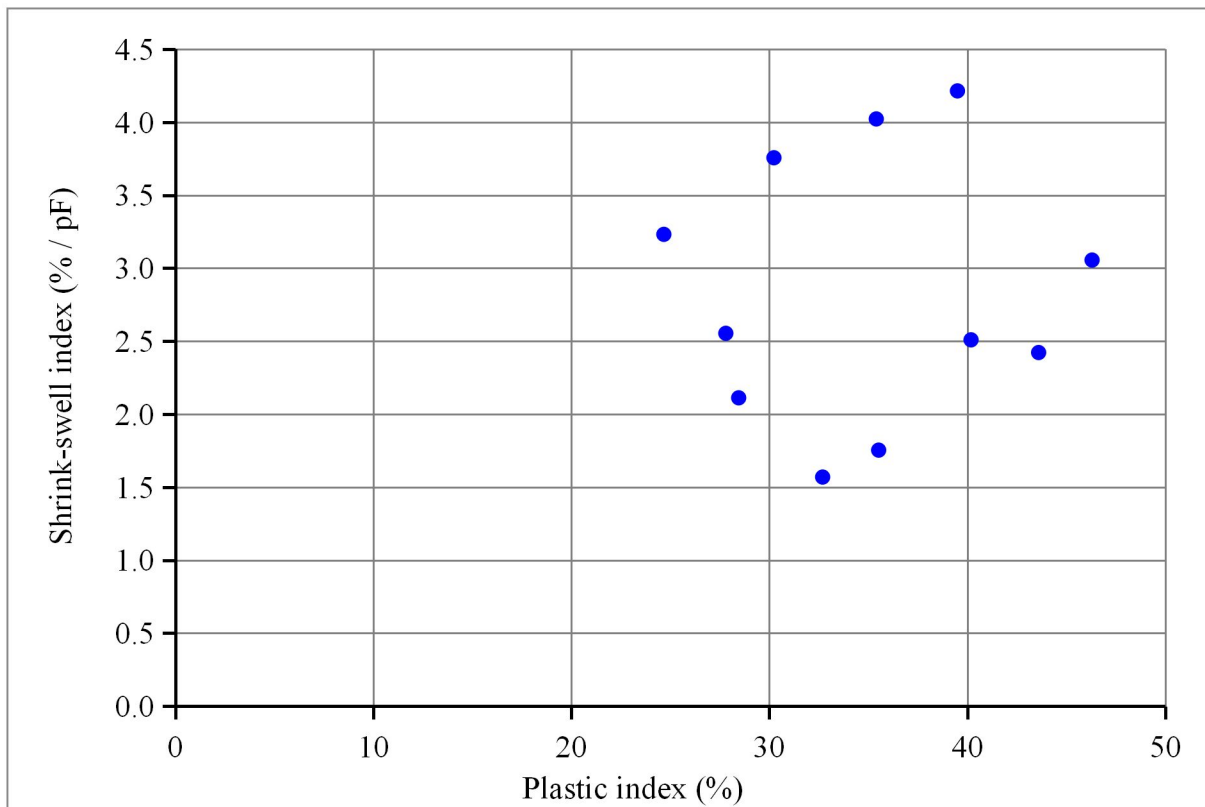


Figure.97 Shrink-swell index vs plastic index (All sediment soil samples)

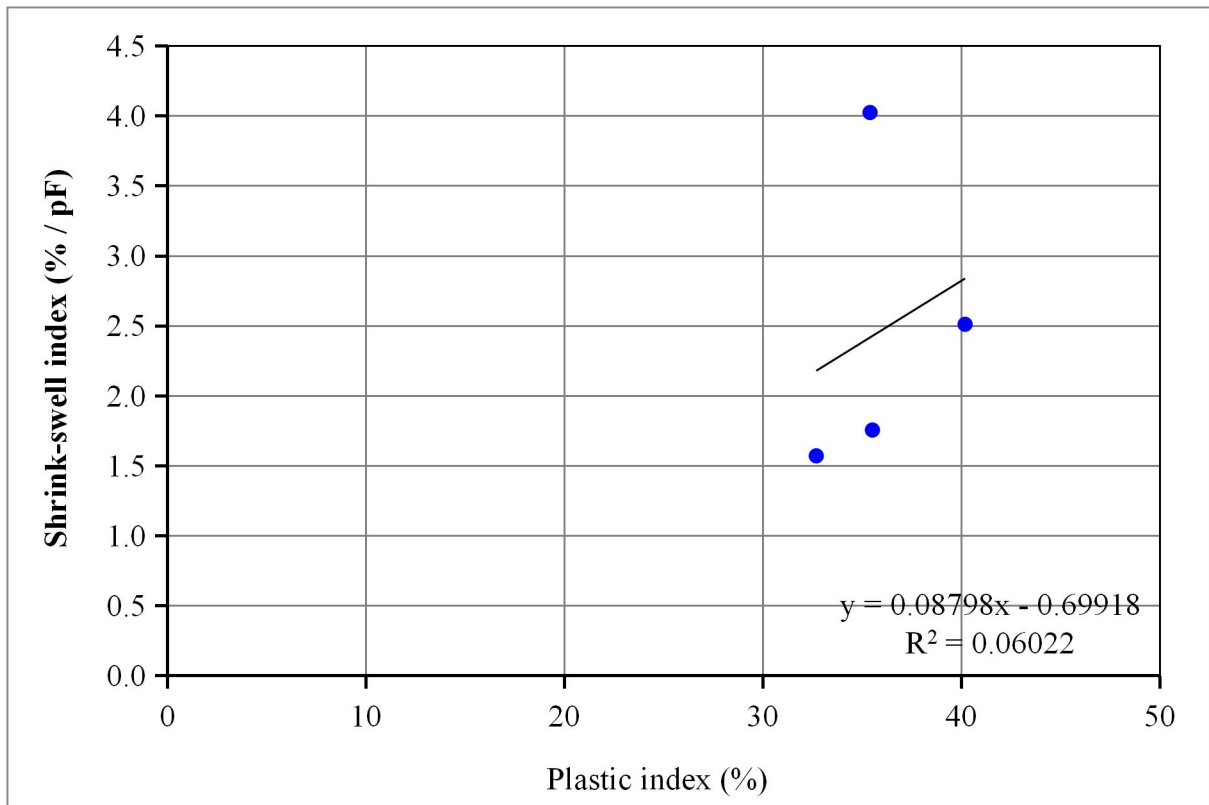


Figure.98 Shrink-swell index vs plastic index (Silurian sediment soil samples)

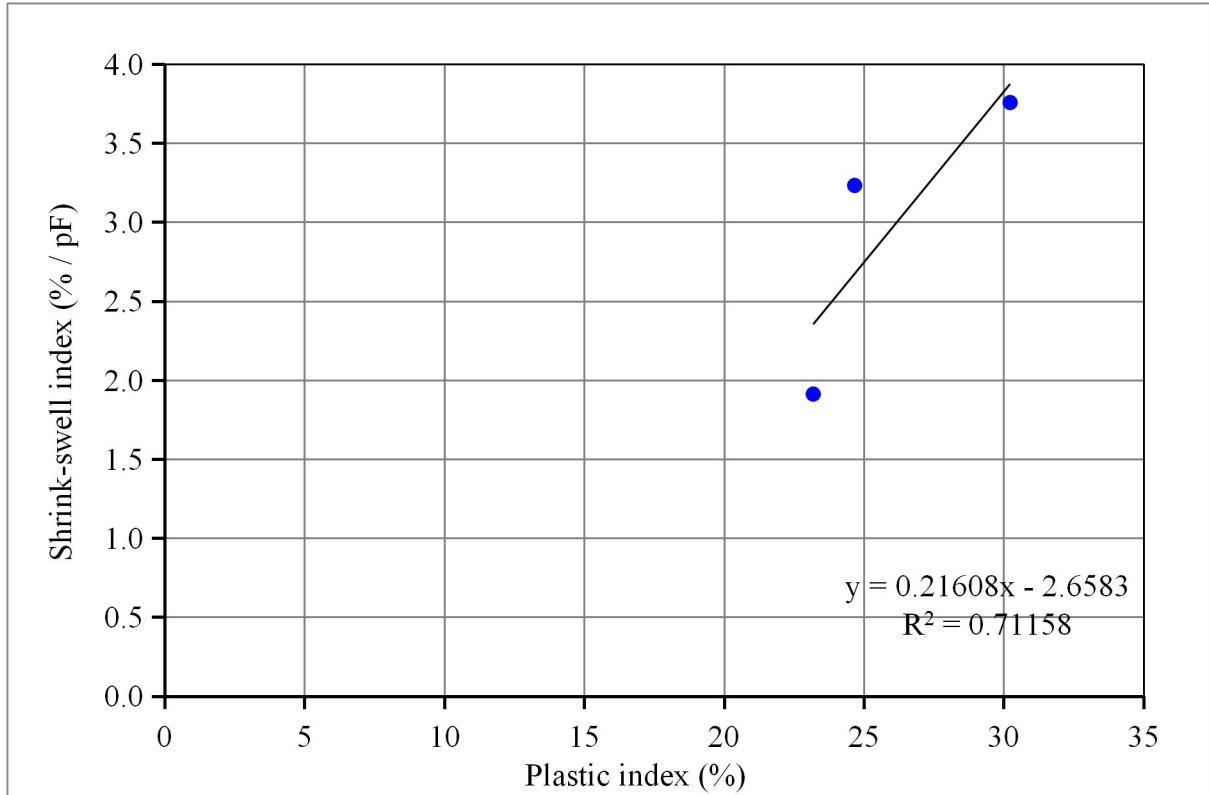


Figure.99 Shrink-swell index vs plastic index (Quaternary alluvium soil samples)

4.5.4 Correlation between the shrink- swell index and Linear Shrinkage

The relationship between the linear shrinkage and the shrink-swell index is plotted in Figure 100.

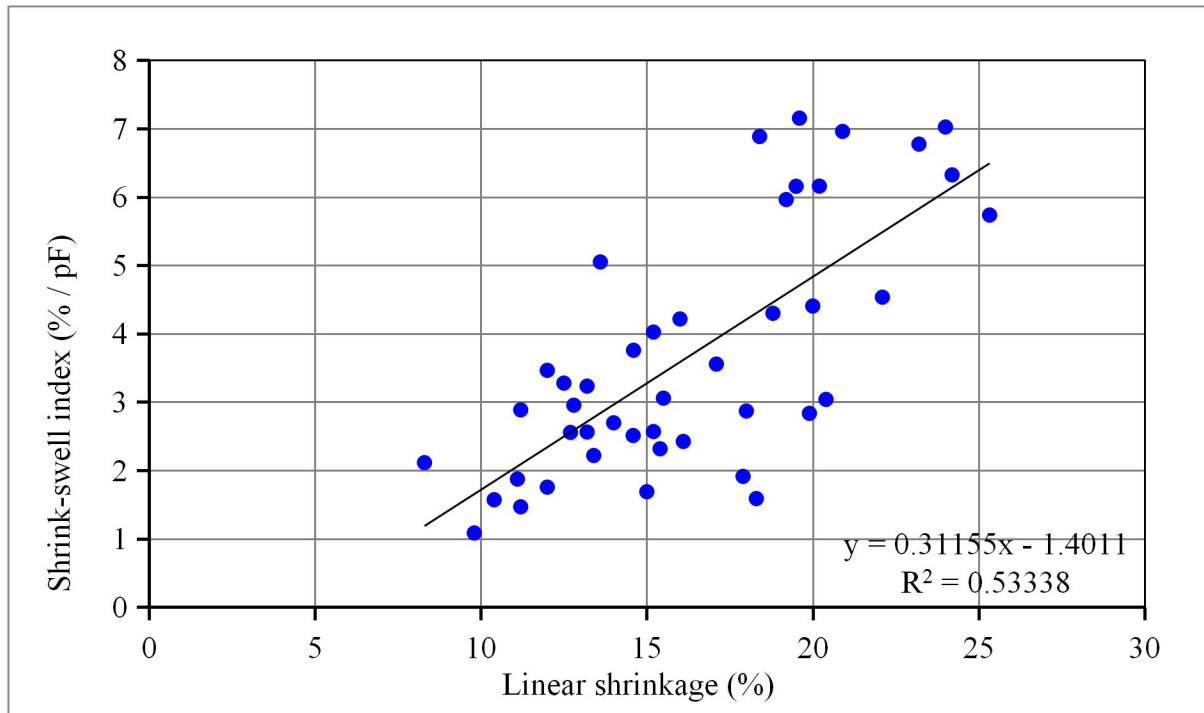


Figure.100 Shrink-swell index vs Linear shrinkage (all soil samples)

To determine if a correlation exists between the shrink-swell index and the linear shrinkage, the values of the shrink-swelling indices are plotted against linear shrinkage shown in Figure 100. The equation of the fitted trend line and R^2 values are:

$$y = 0.3115x - 1.4011 \quad R^2 = 0.5334$$

For a correlation to be considered as a reliable mean for estimating the shrink-swell index of a soil, R^2 of the trend line needs to exceed 0.8. It is apparent that there is a very weak correlation between LS and I_{ss} since the strength of this correlation is only 53.34% (i.e. $R^2 = 0.5334$). Although there is considerable scatter of data, it exhibits an obvious trend between LS and the I_{ss} .

Although the results of the laboratory testing found that traditional index tests were of little general value in assessing the reactivity of expansive soil, the correlation may be confined to a particular soil type. Thus, a number of correlations which were confined to a particular

soil type were attempted in this study. The following analysis is carried out for 2 categories and 5 different types of soil: (a) Tertiary basaltic soil, (b) Quaternary basaltic residual soil, (c) Quaternary basaltic soil, (d) Quaternary alluvium soils and (e) Silurian sediment soil.

4.5.4.1 Basaltic soil

Figure 101 shows the relationship between I_{ss} and LS of all Basaltic soil samples. While there is indeed an obvious trend between the LS and I_{ss} , the correlation is very scattered.

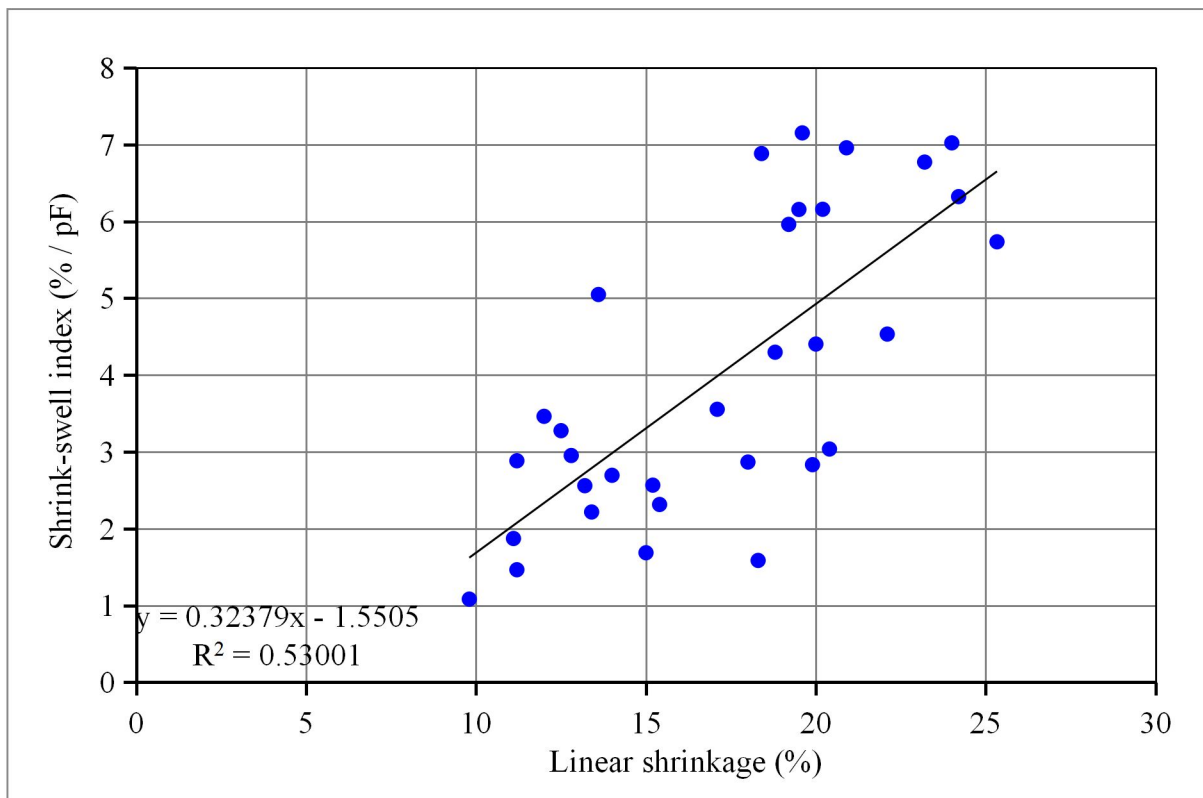


Figure.101 Shrink-swell index vs Linear shrinkage (all basaltic soil samples)

The basaltic soils samples tested in this study can be categorized into three groups, i.e. Tertiary basaltic soil, Quaternary basaltic residual soil and Quaternary basaltic soil. Further attempts are made to determine whether there is a correlation between I_{ss} and LS for each type of basaltic soil. The results are presented in Figures 102 to 104. There is an improved correlation I_{ss} and LS but no clear correlation appeared for Quaternary basaltic soil and Quaternary basaltic residual soil. It should be pointed out that one Quaternary basaltic residual soil sample from Point Cook has a very low value of I_{ss} due to high calcium carbonate content. This sample was not included in the analysis (see Figure 103).

It should be noticed that in [Figure 104](#) there is a strong relationship appeared between I_{ss} and LS for tertiary basaltic soil. The equation of the fitted trend line and R^2 values are:

$$y = 0.8145x - 6.9907 \quad R^2 = 0.9808$$

Although the strength of this correlation is almost 100% (98.08%) fit for the soil test results, it still cannot be used as a reliable predictor for the estimation of the shrink swell index simply because only three samples were considered in the analysis.

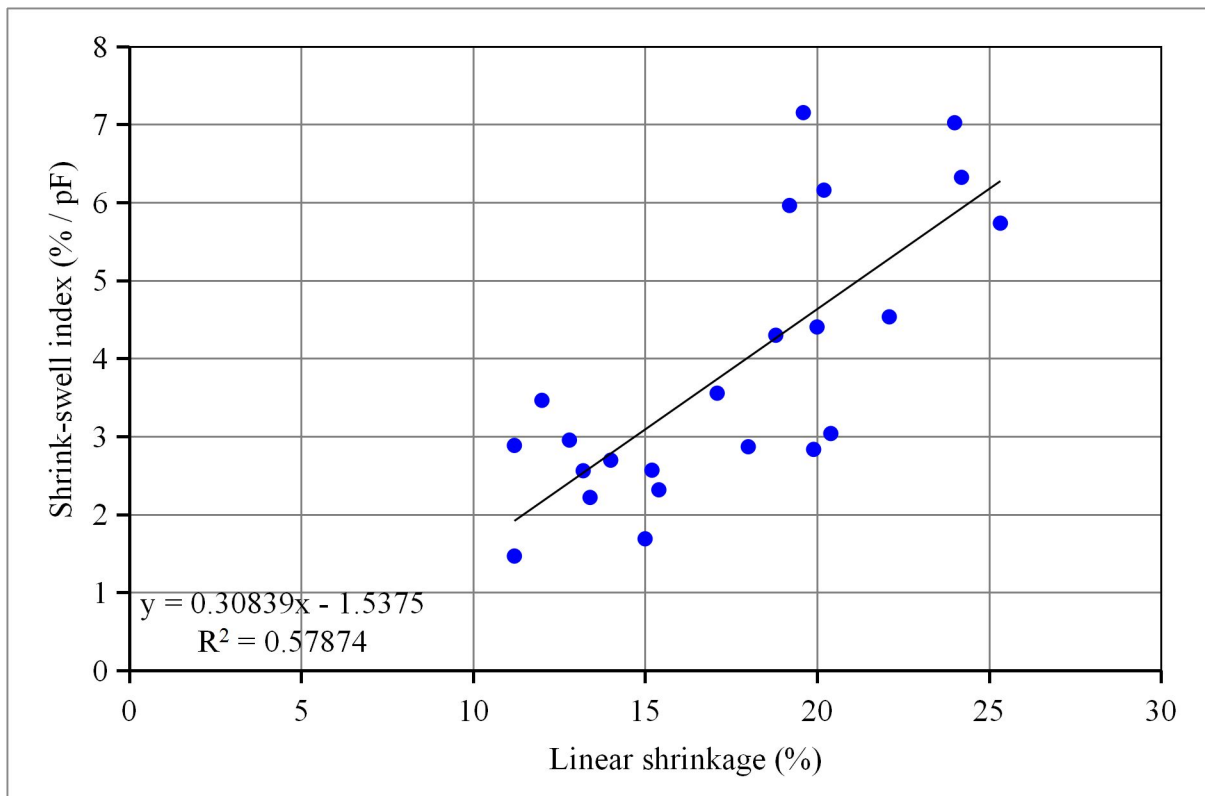


Figure.102 Shrink-swell index vs Linear shrinkage (Quaternary basaltic soil samples)

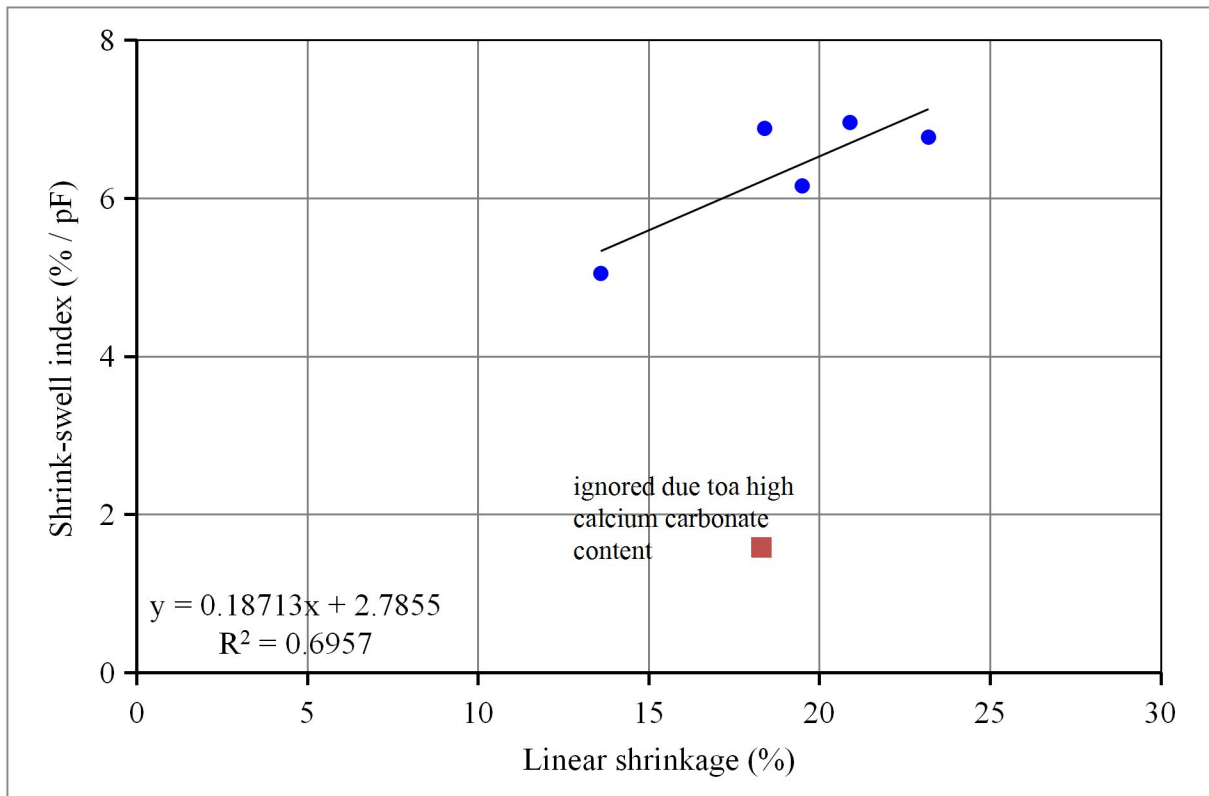


Figure.103 Shrink-swell index vs Linear Shrinkage (Quaternary basaltic residual soil samples)

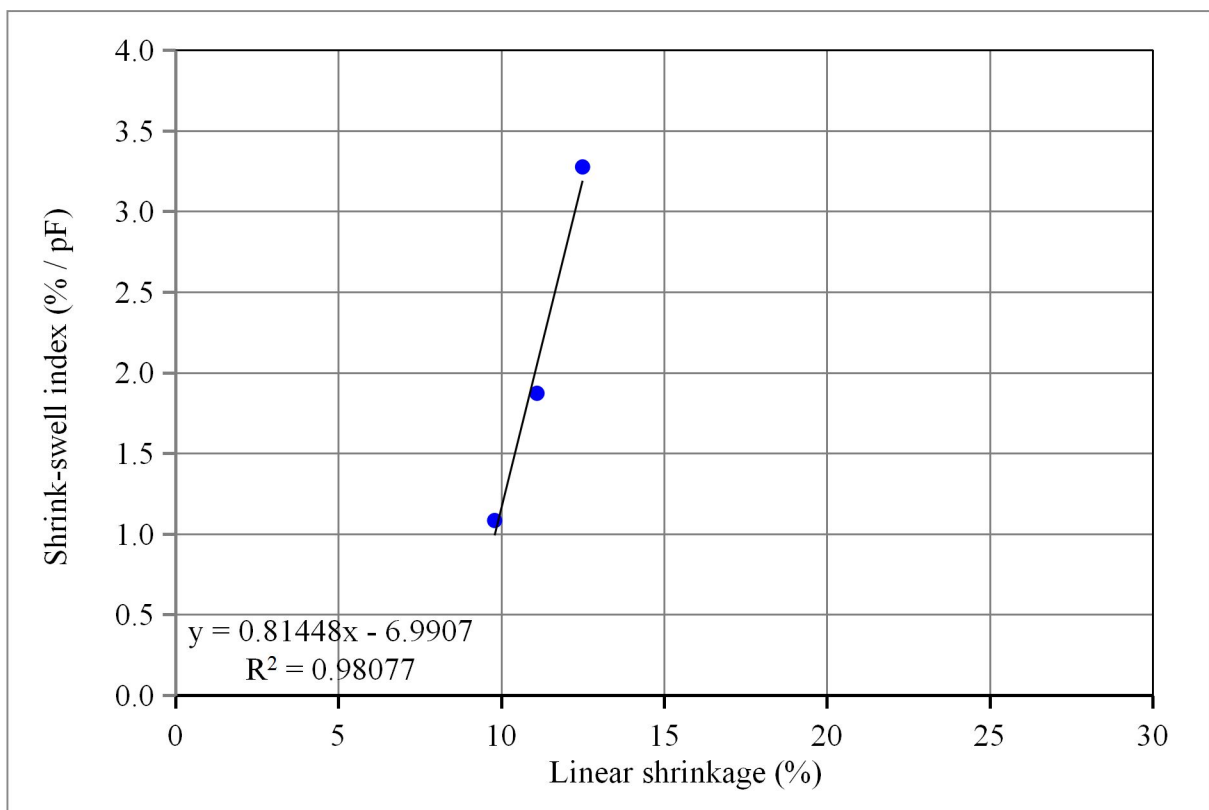


Figure.104 Shrink-swell index vs Linear Shrinkage (Tertiary basaltic soil samples)

4.5.4.2 Sedimentary soil

A similar correlation analysis was also conducted on Sedimentary soil. [Figures 105-107](#) show that significant scatter existed in the relationship between the shrink-swell index and linear shrinkage, so that an unambiguous, reliable relationship does not appear to exist.

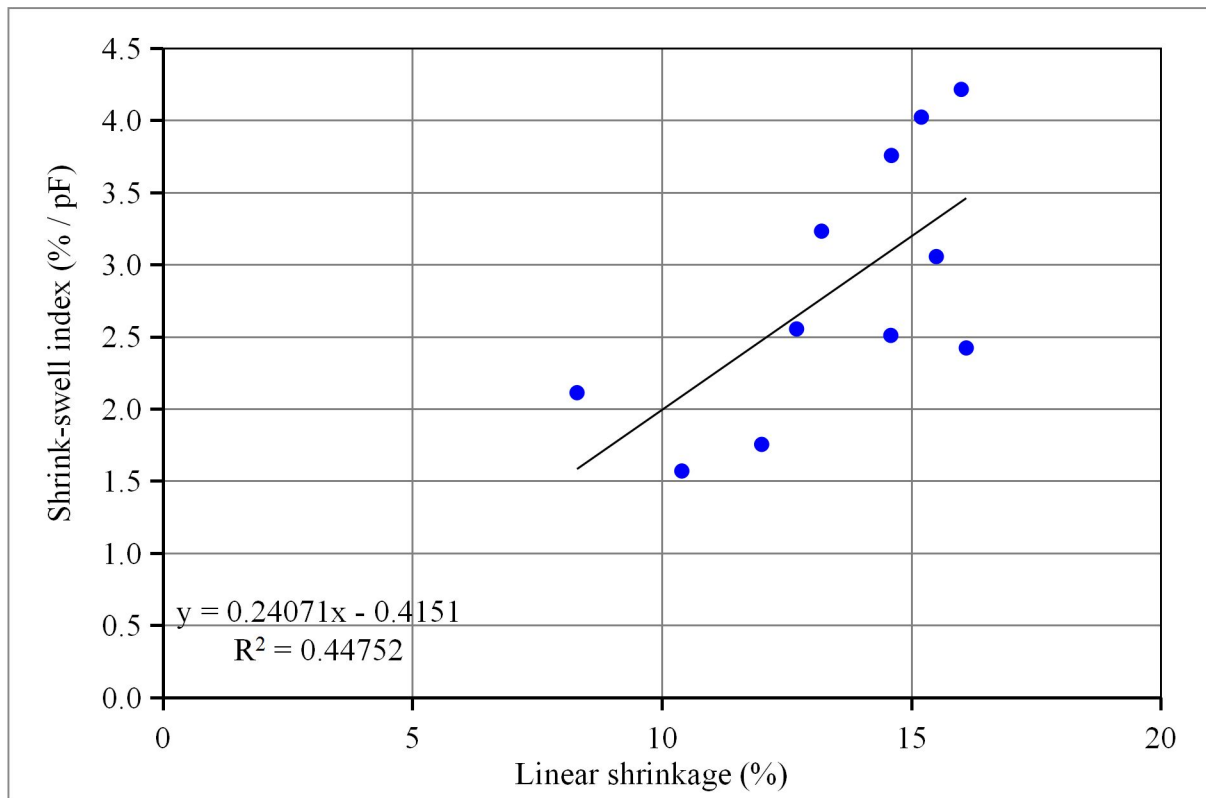


Figure.105 Shrink-swell index vs Linear shrinkage (All sediment soil samples)

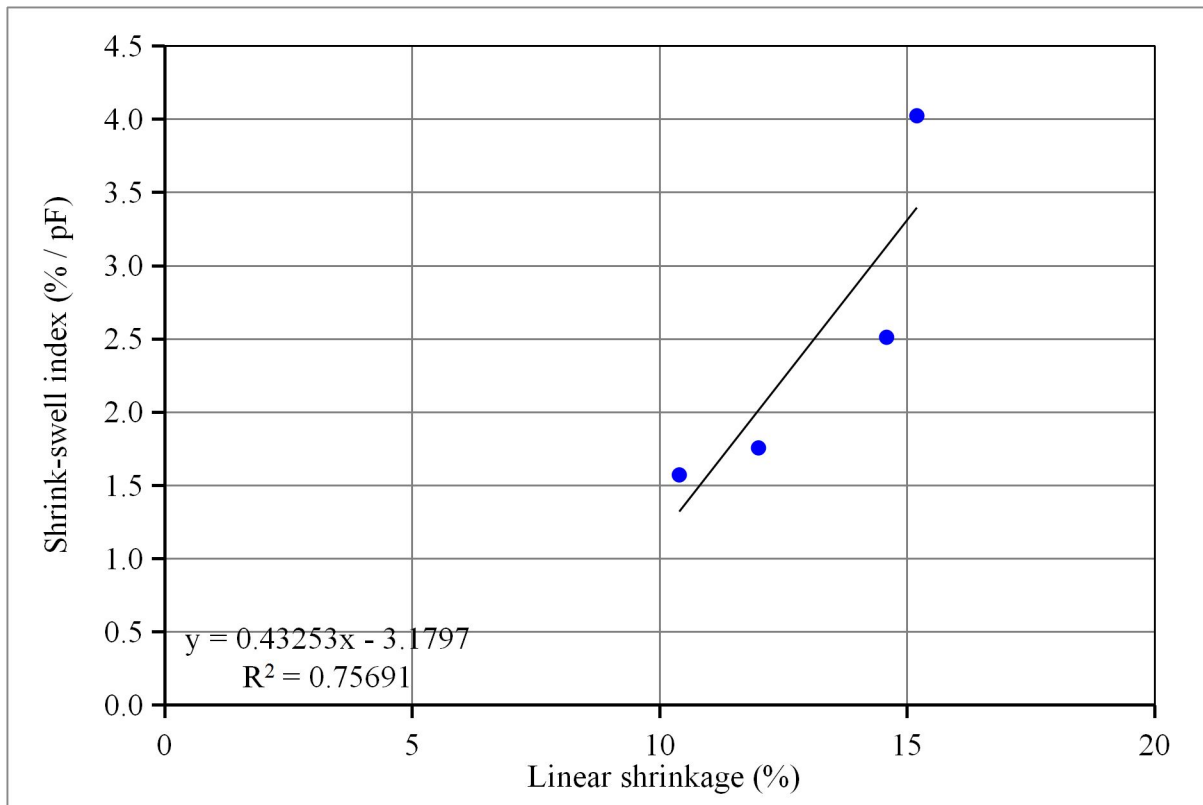


Figure.106 Shrink-swell index vs Linear shrinkage (Silurian sediment soil samples)

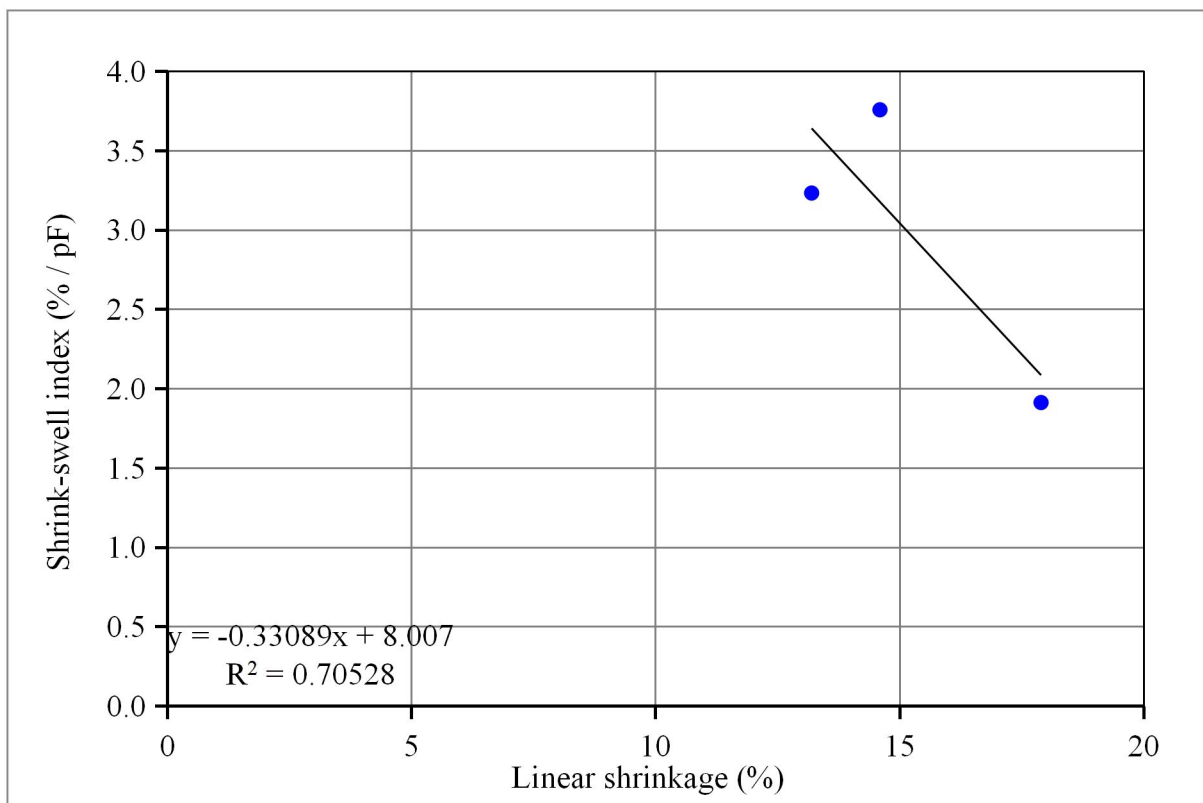


Figure.107 Shrink-swell index vs Linear shrinkage (Quaternary alluvium soil samples)

Table.14 Summary of the range of I_{ss} and average value of traditional soil indices

Atterberg limits Range of I_{ss}	LL (%)	PL (%)	PI	LS (%)
0-2 (%/pF)	50.05	20.69	31.39	13.21
2-4 (%/pF)	50.53	21.42	29.11	14.50
4-6 (%/pF)	67.13	24.74	42.39	18.78
>6 (%/pF)	80.33	27.54	52.79	21.25

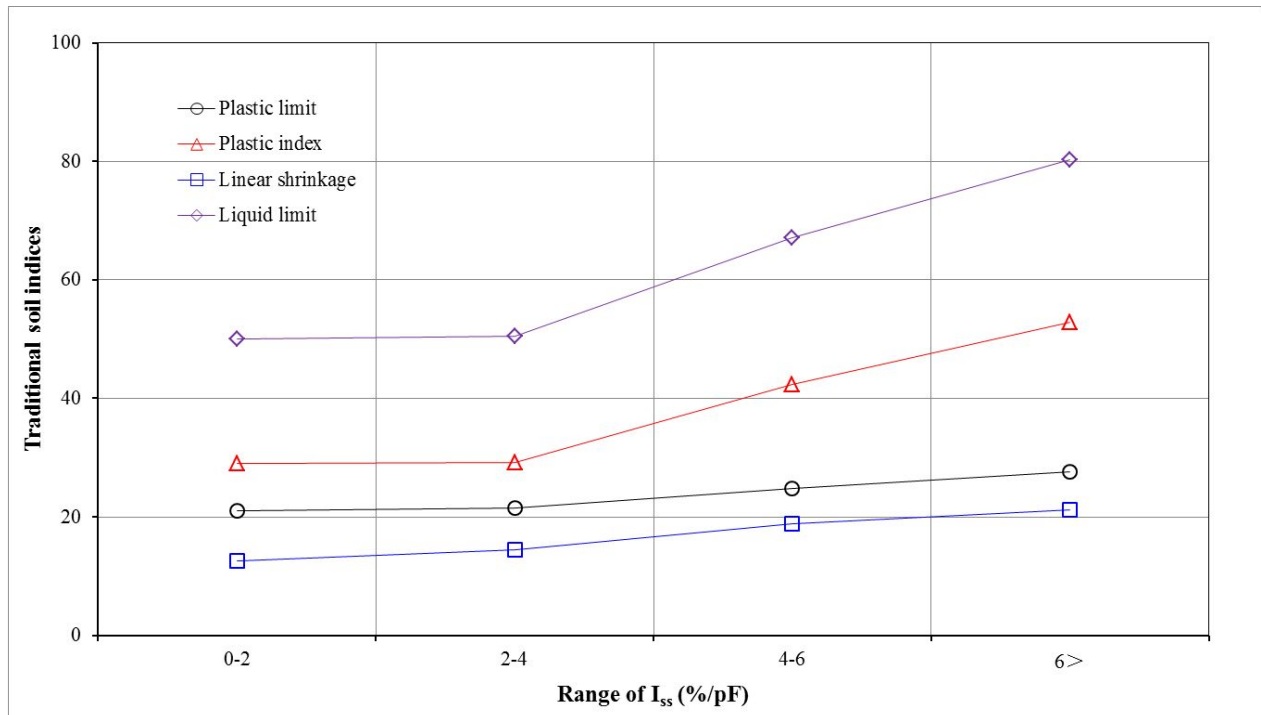
Figure.108 Range of I_{ss} vs average value of traditional soil indices

Table 14 summarises the range of I_{ss} and average values of traditional soil indices. The results are also plotted in Figure 108. Generally, the higher values of Atterberg limit and linear shrinkage, the greater reactivity of soil to moisture change. However the results reported here show that no obvious correlations exist between the shrink-swell index and other traditional soil indices such as liquid limit, plastic limit, plastic index and linear shrinkage.

Study have also proved the point of view of Delaney, Li and Fityus (2005) that traditional index tests were of little general value in assessing the reactivity of expansive soil unless the correlation was confined to a particular soil type. Results show the stronger correlation can be observed from particular soil type. After reviewing the results presented in

the above sections, it seems the linear shrinkage can give more reasonable relationship between soil indices and shrink-swell index.

4.6 Effects of different factors on the results of the shrink-swell test

There are a number of factors that may affect the results of the shrink-swell test. Effect of each factor is discussed in the following sections.

4.6.1 Effect of surcharge pressure on the swelling test

In the swelling test, a surcharge pressure of 25 kPa or the estimated in situ overburden pressure (whichever is greater) is applied at the top of specimen. Highly reactive Braybrook clay was used to study the effect of surcharge pressure on the swelling strain. Three ‘undisturbed’ samples (20 mm high and 50 mm in diameter) from the same depth (0.5 m) with the same initial moisture content (32.45%) were tested under a load of 5, 25 and 50 kPa respectively. The swelling strains are plotted against surcharge pressure in [Figure 109](#). It is apparent that there is a linear relationship between surcharge pressure and swelling strain. The results show that the surcharge pressure has a significant effect on the swelling test. When the load was increased from 25 kPa to 50 kPa, the swelling strain was reduced by approximately 60%.

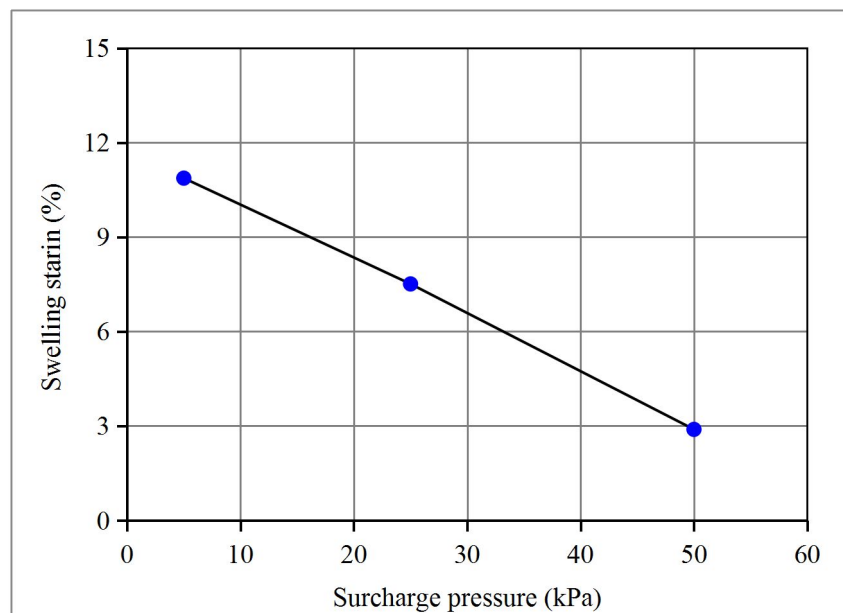


Figure.109 Swelling strain vs surcharge pressure

4.6.2 Effect of sample size on the results of shrinkage and swelling tests

For the swelling test, AS 1289.7.1.1 (2003) requires that soil sample has a minimum diameter of 45 mm. The effect of sample size on the swelling test was evaluated using two different types of soil.

Three ‘undisturbed’ Braybrook clay samples were obtained at a depth of 0.3 m using 20 mm high steel cutting ring with an internal diameter of 38 mm, 45 mm and 63 mm respectively. The swelling tests were conducted in accordance with AS 1289.7.1.1 (2003) and a load of 25 kPa was applied to all three specimens. The relationship of swelling strain vs the diameter of samples is plotted in Figure 110. Due to a high initial water content of 37%, no much swelling was observed. From Figure 110, it can be seen that the swelling strain increased with the diameter of soil samples.

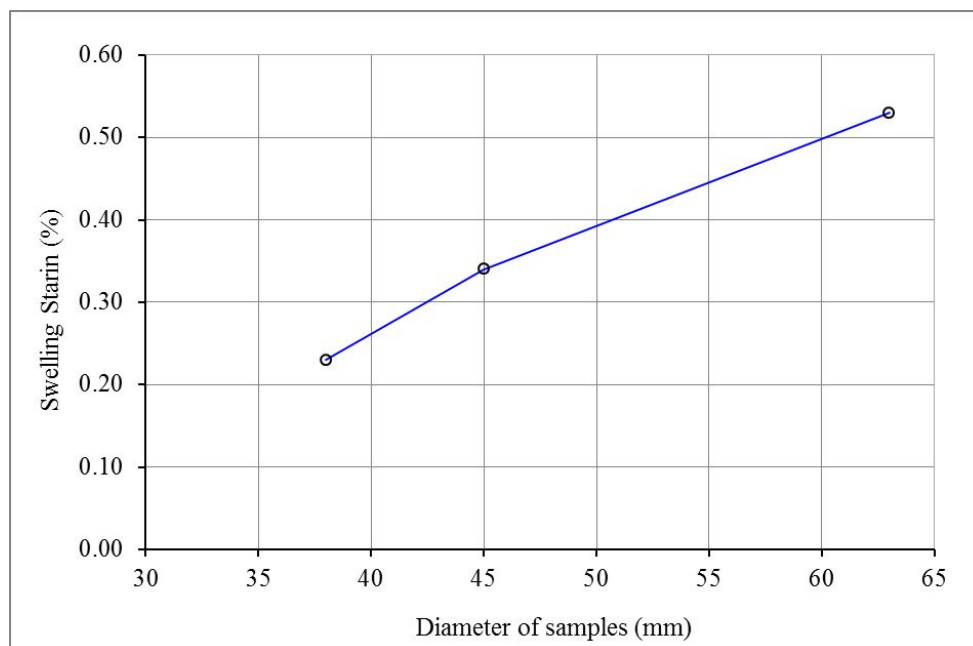


Figure.110 Swelling strain vs diameter of specimen (Braybrook)

The core shrinkage tests were also conducted to evaluate the effect of sample size on the total shrinkage strain. Two Glenroy silt clay samples (reconstituted samples consolidated from slurry) that have a diameter of 38 mm and 50 mm respectively were used. The results are summarized in Table 15. The measured total shrinkage strains are almost same. The diameter of sample seems to have no effect on the shrinkage strain. More tests on different types of soil are needed to investigate the effect of sample size,

Table.15 Total shrinkage strain of Glenroy soil

Diameter of sample (mm)	Initial water content (%)	ε_{sh} (%)
38	24.91	2.82
50	24.56	2.80

4.6.3 Effect of initial moisture content on the results of shrinkage and swelling tests

The adopted experimental approach involved the testing of a number of the same soil from a range of initial moisture contents. Commercial bentonite clay from AMCOL Australia Pty Ltd was selected because it has a very high swelling potential. Figure 111 shows there is a near linear relationship between initial water content and swelling strain. In general, the higher the initial water content, the lower the total swelling potential. The swelling strain reduced by almost 70% when the initial water content of sample was increased from 9% to 33%.

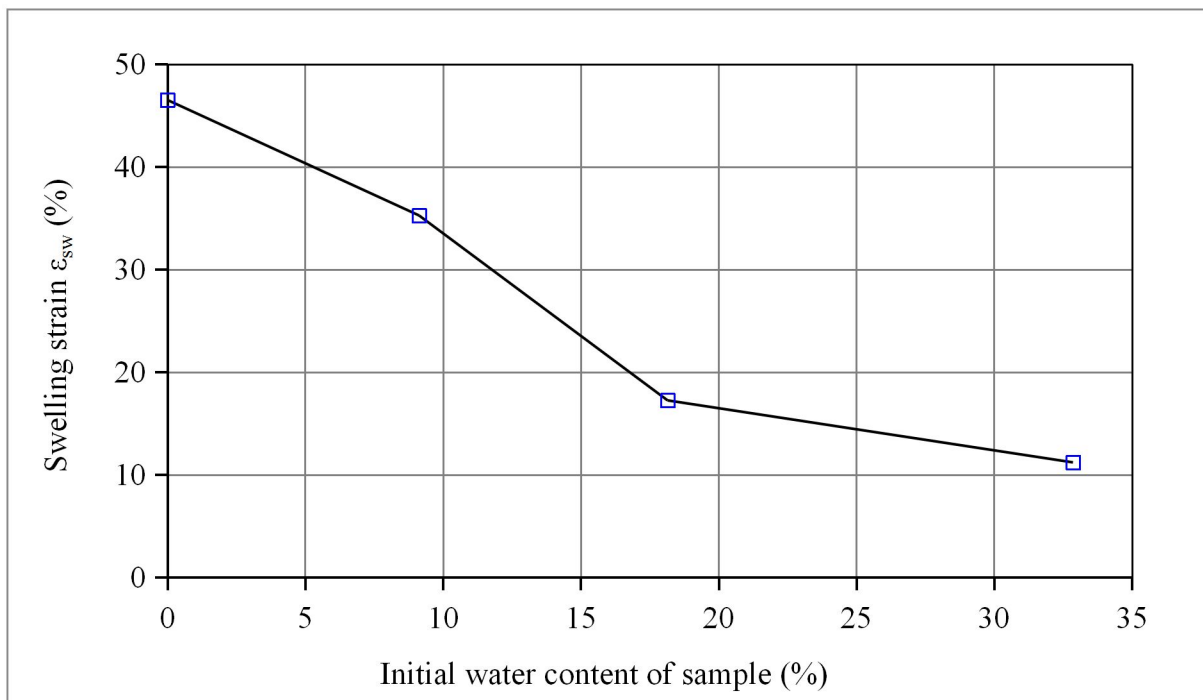


Figure.111 The swell strain vs initial water content of bentonite clay,

The effect of initial moisture content on the results of shrinkage was also investigated using the remoulded Braybrook clay. In order to achieve a soil which would be consistently homogeneous on a test sample scale, the following preparation was performed:

- 1) The soil collected from the field site was first dried/crushed;
- 2) The crushed soil was sieved using a 425 μm screen to remove all coarse sands and gravels.
- 3) The dried soil was mixed to six different initial moisture contents.
- 4) After compacted in a 50 mm diameter tube in 5 equal layers, the tubes were sealed / stored in a horizontal position for two weeks before testing.

During the shrinkage test, the cylindrical core specimens were allowed to air dry slowly for about three week and the shrinkage strain was regularly monitored during this period. The shrink-swell index is plotted against the initial suction and water content of soil samples in [Figures 112-113](#). It is apparent that the initial water content (suction) has a significant impact on the results of the shrink-swell tests. For example, the axial shrinkage strain increased by approximately 40% when the initial water content of soil sample was increased from 24% to 28 % and the initial soil suction was reduced from 4.27 pF to 3.96 pF.

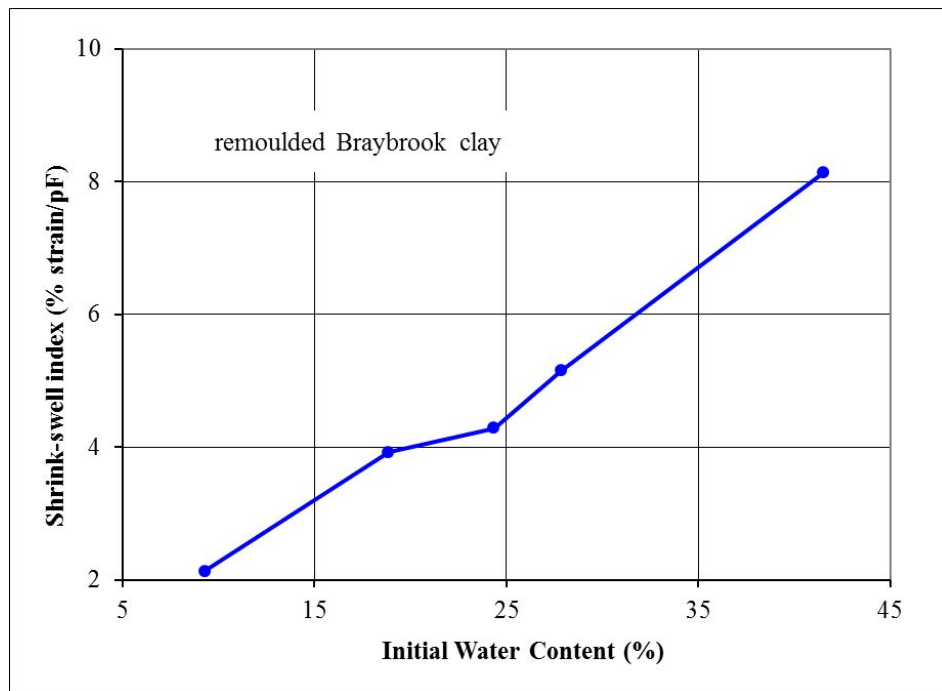


Figure.112 Shrink-swell index vs initial water content of specimen

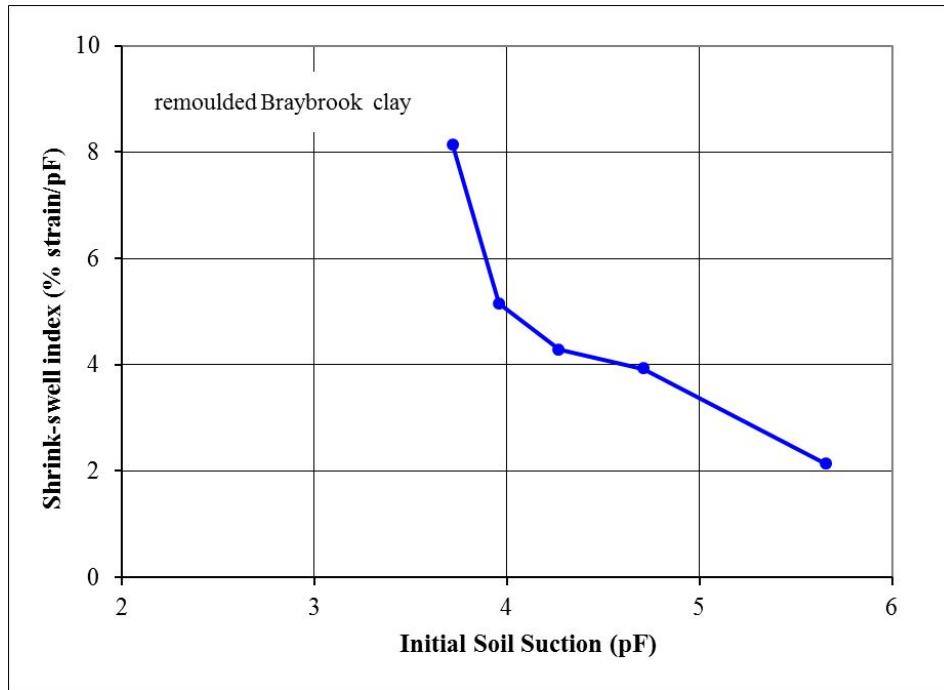


Figure.113 Shrink-swell index vs initial soil suction of specimen)

4.7 X-ray diffraction

Soil minerals play an important role in assessing the soil behaviours. The different minerals in soils could cause large different in soil behaviours. Some minerals have more significant influence on the soil reactivity than others. In this study, clay mineralogy analysis was performed using X-ray diffraction (XRD) analysis to determine the mineral composition of Braybrook and Glenroy soil. The results are shown in [Figure 114 - 115](#). Braybrook soil and Glenroy soil have similar clay mineralogy.

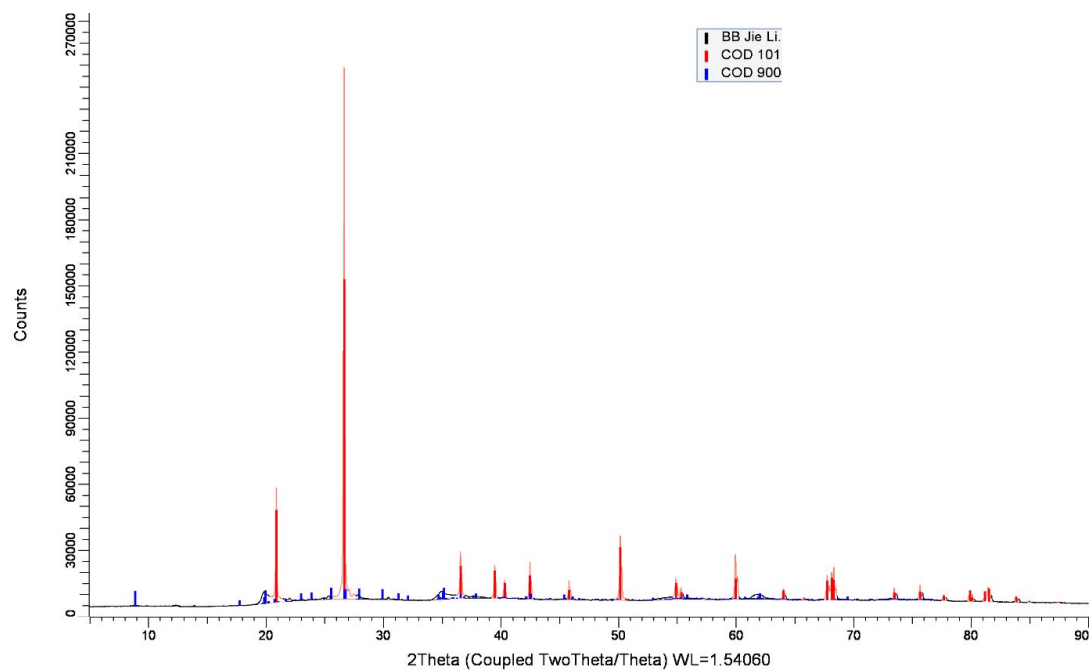


Figure.114 The results of X-ray test for Braybrook

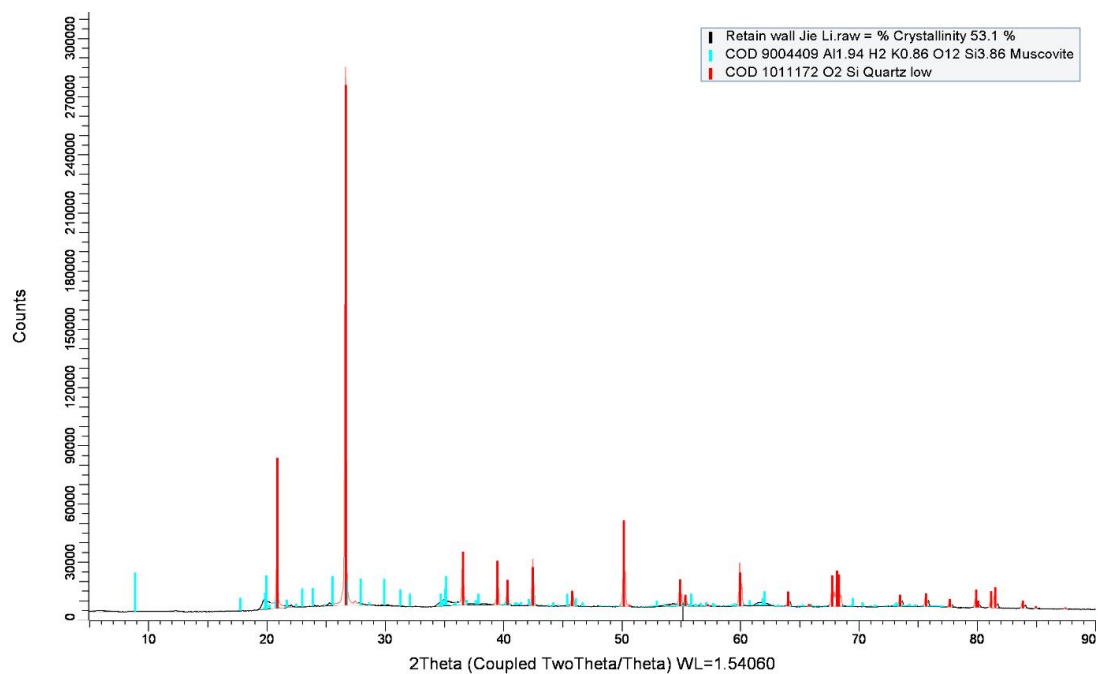


Figure.115 The results of X-ray test for Glenroy

4.8 Summary

This chapter has presented and discussed all the laboratory test results.

First, based on test results, a database of soil information has been built and the results of the shrink-swell tests were used to create an interactive map using Google Maps API which provides information about the nature and distribution of expansive soils in Melbourne metropolitan area. This map/dataset details the location and depth of soil samples collected, site geology information, soil index properties and shrink-swelling indices (I_{ss}).

Secondly, the method/equation for calculation of shrink swell index given in Australian standard was reviewed and evaluated. The appropriateness of using a suction range of 1.8 pF and a lateral restraint factor of 2 for all soils is assessed.

Lateral restraint factor of 2

The results reported here suggest that the values of volume correction factor between 2.6 and 2.9 may be more appropriate and less conservative. It is suggested that more tests on different soils be conducted to assess the appropriateness of the assumed factor of 2.

Suction range of 1.8 pF

The results from this study shows the range of soil suction change Δu from effective saturation to air dry vary between 1.83 pF to 3.22 pF, with a mean value of 2.54 pF. This suggests that the shrink-swell index might be overestimated by approximately 40% and denominator in shrink-swell equation should be 2.5 rather than 1.8.

The results of the core shrinkage testing on the initially saturated reconstituted homogeneous Maryland clay sample shows that 80% of the recorded volume change occurred tween total suction of 2.5 pF and 4.5 pF (an interval of 2 pF) over the 2.3 -5.6 pF (effective saturation to air dry) range (see [Figure 58](#)). This suggests that a value of 1.8 pF is not unseasonable although a value of 2 may be more appropriate and less conservative.

Thirdly the correlation between the shrink-swell index and traditional soil indices shows a higher value of Atterberg limit generally indicate greater reactivity of soil to moisture change. However the results reported in this study show that no obvious correlations exist between the shrink-swell index and other traditional soil indices such as liquid limit, plastic limit, plastic index and linear shrinkage

Finally, the impact of surcharge loading, sample size and initial soil moisture (suction) on the shrink-swell test were also assessed. The results show that

- 1) The surcharge pressure has a significant effect on the swelling test. When the load was increased from 25 kPa to 50 kPa, the swelling strain was reduced by approximately 60%.
- 2) An increase in the diameter of soil samples led to an increase in the total swelling strain but had no effect on the total shrinkage strain.
- 3) A lower initial soil suction or a higher initial water content of sample led to a higher value of the shrink-swell index.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

In order to provide information about the nature and distribution of expansive soils in Melbourne Metropolitan Areas, a series of laboratory tests including shrink-swell test, soil-water characteristic curve, liquid limit, plastic limit, linear shrinkage, soil suction measurement and X-ray diffraction (XRD) analysis, have been performed on soil samples collected from 47 different field sites across 37 suburbans of Victoria which cover a wide range of soil, geological and geographic conditions. The study indicates that highly reactive soils are mainly presented in the Western and Northern suburbs of Melbourne, and many areas have a shrink-swell index higher than 6%/pF. The results of the laboratory tests were used to establish a library of shrink-swell indices. An interactive map showing the distribution and reactivity of Melbourne soils has been developed, which can be used in routine practice by local engineers to assess/calibrate the shrinkage indices estimated on the visual-tactile basis.

In this thesis, a number of experiments have been conducted to assess the empirical equation of AS1289.7.1.1 (2003) and its assumptions (i.e. an empirical correction factor of 2 for axial swelling test and assumed suction range of 1.8 pF). The results of three-dimensional volumetric shrinkage tests suggest that the values of correction factor between 2.6 and 2.9 are likely to be appropriate. More experiments on different soils should be conducted to further assess the appropriateness of the assumed factor of 2. Statistical analysis of 24 shrink-swell tests reveals the range of soil suction change Δu from effective saturation to air dry vary between 1.83 pF to 3.22 pF, with a mean value of 2.54 pF. This suggests that a value of 2.5 may be more appropriate and less conservative.

A series of laboratory tests have also been carried out to evaluate the effects of surcharge pressure, sample size and initial water content on shrink and swell tests. The results show:

- An increase in surcharge pressure led to a decrease in swelling strain;
- The swelling strain increased slightly with the diameter of soil samples;

- The higher the initial water content or the lower the initial soil suction, the higher the I_{ss} values obtained from the shrink-swell tests.

In addition, a number of correlations were attempted in this thesis between the shrink-swell test and other soil tests including linear shrinkage, plastic index and liquid limit. The results show that there is no obvious correlation between the shrink-swell index and traditional soil indices such as liquid limit, plastic limit, plastic index and linear shrinkage.

Overall, the outcome of this research can lead to a better understanding of shrink-swell behaviour of Melbourne soils and subsequently improve current site classification and footing design.

5.2 Recommendations for Further Work

It is recommended that further research be carried out in the following areas:

- Due to time limitation, only 60 shrink swell tests were conducted. More tests are required for the database and the interactive soil map also needs further input and analysis.
- More experiments are required to further assess the appropriateness of the correction factor of 2 and the assumed suction range of 1.8 pF.
- Further study of effects of sample size and initial water content on the results of the shrink-swell index is also needed.

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